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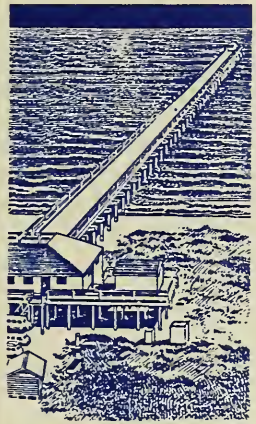

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COASTAL ENGINEERING RESEARCH CENTER

TECHNICAL REPORT CERC-85-10

WAVE STABILITY TESTS OF DOLOS AND STONE REHABILITATION DESIGNS FOR THE EAST BREAKWATER, CLEVELAND HARBOR, OHIO

Experimental Model Investigation

by

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Final Report

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20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

An experimental model investigation was conducted using a two-dimensional stability model at an undisturbed linear scale of 1:28.5 (model to prototype). The purposes of the stability tests were as follows:

a. Evaluate the stability of the proposed 4-ton dolos trunk section (Plan 1) when exposed to design wave and still-water level (swl) conditions.

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20. ABSTRACT (Continued).

b. Determine the degree of breakwater damage that could occur in Plan 1 for a storm condition that exceeds the design condition.

c. Determine the maximum nonbreaking wave heights for which the existing 2-ton dolos design (Plan 2) and the proposed 4-ton dolos and 9- to 20-ton armor stone designs (Plan 3) could be considered to be adequately designed.

Test results indicated that Plan 1 was only marginally acceptable and would most likely require significant amounts of maintenance if exposed to design level wave conditions (7.0- to 9.0-sec, 13.4-ft nonbreaking waves at an swl of +4.9 ft low water datum (lwd)). Plan 1 also proved to be a very inadequate design for wave conditions exceeding the design conditions (7.0- to 9.0-sec, 15.0-ft nonbreaking waves at an swl of +4.9 ft lwd). Plans 1, 2, and 3 proved to be adequate designs for 7.0- to 9.0-sec, 12.0-, 10.5-, and 13.4-ft nonbreaking waves, respectively, at an swl of +4.9 ft lwd.

PREFACE

The model investigation reported herein initially was requested by the US Army Engineer District, Buffalo (NCB), in a letter to the US Army Engineer Waterways Experiment Station (WES) dated 21 December 1984. Funding authorizations by NCB were granted in Intra-Army Orders NCB-IA-85-26JD dated 20 December 1984 and NCB-IA-85-41JD dated 13 March 1985.

Model tests were conducted at WES under the general direction of Dr. R. W. Whalin, former Chief of the Coastal Engineering Research Center, Mr. C. E. Chatham, Chief of the Wave Dynamics Division, and Mr. D. D. Davidson, Chief of the Wave Research Branch. Tests were conducted by Mr. W. G. Dubose, Engineering Technician, under the supervision of Mr. D. G. Markle, Research Hydraulic Engineer. This report was prepared by Messrs. Markle and Dubose. Mr. C. C. Calhoun, Jr., was Acting Chief of CERC during the preparation and publication of this report. This report was edited by Mrs. Beth F. Vavra, Publications and Graphic Arts Division.

Liaison was maintained with Mr. Denton Clark, Chief of the Coastal Engineering Section, NCB, during the course of this study by means of conferences, progress reports, and telephone conversations.

Director of WES was COL Allen F. Grum, USA. Technical Director was Dr. Robert W. Whalin.

CONTENTS

	Page
PREFACE	1
CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT	3
PART I: INTRODUCTION	5
Background	5
Purposes of Model Study	5
PART II: THE MODEL	7
Design of Model	7
Test Facilities and Equipment	8
Model Construction and Testing Procedures	8
PART III: TESTS AND RESULTS	11
Plan 1, 4-ton Dolosse	11
Plan 2, 2.3-ton Dolosse	14
Plan 3, 9- to 20-ton Armor Stone	15
PART IV: CONCLUSIONS	17
PART V: DISCUSSION	18
TABLES 1-4	
PHOTOS 1-43	
PLATES 1-3	

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENTS

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
miles (US statute)	1.609347	kilometres
pounds (force)	4.448222	newtons
pounds (force) per cubic foot	157.087467	newtons per cubic metre
tons (force)	8.896444	kilonewtons

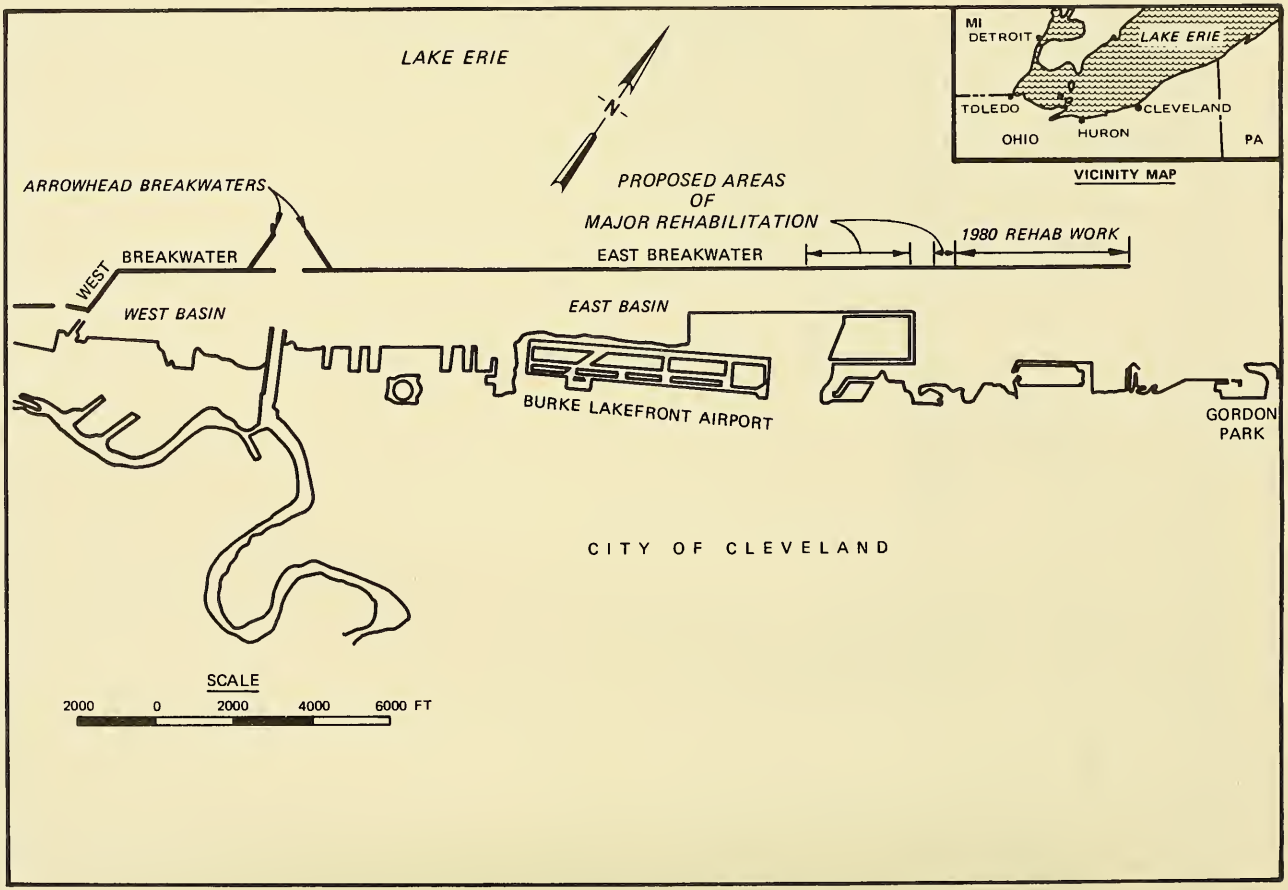

Figure 1. Location and vicinity map

WAVE STABILITY TESTS OF DOLOS AND STONE REHABILITATION DESIGNS FOR THE EAST BREAKWATER, CLEVELAND HARBOR, OHIO

Experimental Model Investigation

PART I: INTRODUCTION

Background

1. Cleveland Harbor is located at Cleveland, Ohio, about 110 miles* east of Toledo, Ohio, and about 191 miles west of Buffalo, New York (Figure 1). The harbor is protected by a 20,970-ft east breakwater, 6,048-ft west breakwater, and two 1,250-ft arrowhead breakwaters. The easterly 17,970 ft of the east breakwater is a rubble-mound structure with a keyed-and-fitted system of specially shaped armor stone. Using construction techniques and armor stone similar to the original construction, the east breakwater was repaired on numerous occasions between 1927 and 1978. In 1980, the eastern 4,400 ft of the east breakwater was rehabilitated. Two layers of 2-ton unreinforced dolosse were placed on the lakeside of the trunk and around the head. A total of 29,700 dolosse were used during the original construction and 200 additional dolosse were used to repair damage that occurred on the breakwater head during the storm of April 1982. A survey in April 1984 reported that 659 of the 29,900 dolosse had been broken and remained on the structure.

2. At the present time, the US Army Engineer District, Buffalo (NCB), is planning the rehabilitation of an additional 3,300 ft of the east breakwater trunk (Figure 1). Due to breakage observed on the original 2-ton dolos rehabilitation, NCB is proposing the use of either 4-ton dolosse or a 9- to 20-ton armor-stone mix on the new rehabilitation work.

Purposes of Model Study

3. The purposes of this two-dimensional (2-D) breakwater stability study were to:

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

a. Evaluate the stability of the proposed 4-ton dolos trunk section when exposed to design wave and still-water level (swl) conditions specified by NCB.

b. Determine the degree of breakwater damage that could occur on the 4-ton dolos design for a storm condition (specified by NCB) that exceeds the design wave conditions.

c. Determine the maximum nonbreaking wave heights for which the existing 2-ton dolos design and the proposed 4-ton dolos and 9- to 20-ton armor-stone designs could be considered adequately designed.

PART II: THE MODEL

Design of Model

4. Tests were conducted at an undistorted linear scale of 1:28.5, model to prototype. Scale selection was based on the size of model dolosse and armor stone available relative to the size of the proposed prototype dolosse and armor stone, elimination of stability scale effects,* and capabilities of the available test flume. Based on Froude's model law** and the linear scale of 1:28.5, the following model-to-prototype relations were derived (dimensions are in terms of length (L) and time (T)):

Characteristic	Dimension	Model-Prototype Scale Relations
Length	L	L_r = 1:28.5
Area	L^2	$A_r = L_r^2$ = 1:812.3
Volume	L^3	$V_r = L_r^3$ = 1:23,149.1
Time	T	$T_r = L_r^{1/2}$ = 1:5.3

5. The specific weights of the model construction materials differed from their prototype counterparts; therefore sizing of the model construction materials was based on the following transference equation:

$$\frac{\left(W_a\right)_m}{\left(W_a\right)_p} = \frac{\left(\gamma_a\right)_m}{\left(\gamma_a\right)_p}\left(\frac{L_m}{L_p}\right)^3\left[\frac{\left(S_a\right)_p - 1}{\left(S_a\right)_m - 1}\right]^3$$

where

subscripts m, p = model and prototype quantities, respectively

W_a = weight of an individual armor unit or stone, lb

γ_a = specific weight of an individual armor unit or stone, pcf

L_m/L_p = linear scale of the model

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* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, US army Engineer waterways Experiment Station, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manual on Engineering Practices No. 25, American Society of Civil Engineers, New York.

S_a = specific gravity of an individual armor unit or stone relative to the water in which the breakwater is constructed, i.e., $S_a = \gamma_a/\gamma_w$

γ_w = specific weight of water, pcf

Test Facilities and Equipment

6. All of the 2-D breakwater stability tests (incident wave crests were parallel to the longitudinal axis of the breakwater) were conducted in a 6.75-ft-wide, 4-ft-deep, and 119-ft-long concrete flume. The test facility is equipped with a vertical displacement wave generator capable of producing monochromatic waves of various periods and heights.

Model Construction and Testing Procedures

7. Based on prototype information showing that bathymetry lakeward of the east breakwater is quite flat and the fact that model tests would be conducted with nonbreaking waves, it was agreed by both NCB and the US Army Engineer Waterways Experiment Station (WES) that model tests would use a flat-bottom bathymetry and a breakwater toe elevation of -30.0 ft low water datum (lwd).

Selection of test conditions

8. An swl of +4.9 ft lwd and wave periods of 7, 8, and 9 sec were selected by NCB for use with a 13.4-ft design wave height and a 15.0-ft wave height. The 15.0-ft wave height was selected as an extreme high-wave condition for which NCB wanted to know the degree of damage to a 4-ton dolos design. These conditions were combined into a design storm, Hydrograph A (Table 1), and an extreme event, Hydrograph B (Table 2). Additional test conditions are described in PART III: TEST AND RESULTS.

Flume calibration

9. Prior to installation of the breakwater test section, the flume was calibrated for the wave and swl conditions described in paragraph 8. Test waves of the required characteristics were generated by varying the amplitude and frequency of the wave generator paddle motion. Changes in water-surface elevations with time (wave heights) were measured with an electrical wave gage positioned in the flume where the lakeside toe of the breakwater would be located.

Method of constructing test sections

10. The typical existing east breakwater cross section supplied by NCB (Plate 1) was constructed to reproduce as closely as possible the existing breakwater construction. Core material was dumped by bucket or shovel, smoothed to grade, and compacted with hand trowels to simulate consolidation that has occurred due to wave action. The core was covered with one layer of specially shaped laid-up stone. Armor stones were placed one at a time in an effort to obtain the existing keyed-and-fitted construction. The bedding, berm, and underlayer rehabilitation materials, designated by NCB for their respective existing or proposed rehabilitation designs, were sequentially placed and smoothed to grade on the lakeside of the existing breakwater section in a manner that reproduced usual construction methods. The lakeside slope then was covered with two layers of randomly placed dolosse or armor stone, depending on the plan being tested.

Model operation

11. After "before-test" photographs were taken, the flume was flooded to an appropriate depth and the structure was exposed to shakedown and test wave conditions. Shakedown waves allowed some natural settling and nesting of the newly constructed section that would occur under lower level wave conditions prior to being exposed to a design level storm. Prototype test time was accumulated in 30-sec (model time) cycles, i.e., the wave generator was started, run 30 sec, and then stopped. This procedure prevented contamination of incident waves by waves rereflected from the wave generator. After each 30-sec cycle, sufficient time was provided for the flume to still out before the next cycle was run. During stilling time between cycles, detailed model observations of the structure's response to the previous cycle of test waves were recorded by the model operator. Observations included any movement occurring on the structure and a general statement of the structure's condition at that point in the test. All test conditions were run for at least the durations indicated for each hydrograph step. Where damage did not stabilize during the normal duration of the hydrograph step, the test condition duration was extended until damage had stabilized or the damage level exceeded an acceptable amount. At conclusion of the hydrograph, the flume was drained and the after-test condition of the structure was summarized in test notes and documented with photographs. Where test hydrographs were run back-to-back, the flume was refilled with water and the structure was exposed to conditions

of the second hydrograph. The same test procedures used during the first hydrograph were used to accumulate test time and document structure changes that occurred during the second hydrograph. At the conclusion of the test, the cummulative response of the structure to both hydrographs was summarized in test notes and documented with photographs. The dolos or armor-stone layers then were removed, underlayer stone was straightened as needed, and the armor units once again were placed on the structure and the test was repeated. The purpose of the repeat test was to determine the presence of any uncontrolled variations in model construction technique that might affect stability of the structure.

Methods of reporting model observations and test results

12. The following list of adjectives, in order of increasing severity, was used for recording model observations and reporting test results of damage for each test section: (a) slight, (b) minor, (c) moderate, (d) significant, (e) major, and (f) extensive. Slight and minor were used to describe acceptable results, moderate described borderline acceptability, while significant to extensive described unacceptable conditions of increasing severity. Use of these adjectives allowed some quantification of the severity and/or amount of rocking in place, reorientation and displacement of the primary armor units, wave overtopping, and resulting damage accrued by the breakwater's primary cover-layer units.

PART III: TEST AND RESULTS

Plan 1, 4-ton Dolosse

13. The proposed 4-ton dolos rehabilitation section (Plan 1, Plate 1 and Photos 1 and 2) was a two-layer system of randomly placed dolosse on a 1V-on-2H lakeside slope. The dolosse coverage extended from a 13-ft-wide crown (elevation +10.3 ft lwd) to a toe berm elevation of -22.0 ft lwd. Plan 1 was exposed to the wave and swl conditions of Hydrograph A (Table 1). During Step 1, minor to moderate rocking of several dolosse was observed around the swl and on the upper slope and crown. Two dolosse were displaced from the dolos crown area onto the old laid-up stone crown and one of these dolos then was displaced onto the harbor-side slope during a later cycle of Step 1. It also was noted that the combined upslope packing and downslope consolidation of the dolos armor around the swl created an area of dolos separation around the swl. In this area, the dolos armor porosity was much higher than other areas of the structure and the underlayer had become very visible but was showing no signs of movement. Four additional dolosse were displaced from the crown onto the harbor-side slope and one dolos originally placed below the swl was displaced down the lakeside slope during Step 2. The severity of dolos rocking and number of units rocking seemed to increase slightly at the start of this step, but appeared to decrease in severity by the end of Step 2. The wave conditions of Steps 1 and 2 produced moderate to significant overtopping while major overtopping was observed during Step 3 of Hydrograph A. It appeared that the 9-sec waves were not impacting as hard on the dolos, but that most of the wave energy was passing over the breakwater crown and was being dissipated on the harbor-side of the structure. No additional displacement occurred during this step and the amount of dolos rocking appeared to have decreased from that observed during the previous step. At the end of Hydrograph A, the dolos armor showed minor spot damage on the breakwater crown, slight localized damage on the lower lakeside slope, and moderate amounts of dolos reorientation around the swl (Photos 3 and 4). The dolos reorientation around the swl was the result of the upslope packing and downslope consolidation of dolosse that occurred during Step 1 and did not seem to increase in severity during Steps 2 and 3 of Hydrograph A.

14. The structure showed only minor to moderate rocking of a few

dolosse on the crown and upper lakeside slope during Step 1 of Hydrograph B (Table 2). The dolosse on the breakwater crown sustained significant damage during Step 2 of Hydrograph B. Eight dolosse were displaced from the crown down onto the harbor-side slope, 2 dolosse were displaced on the lower lakeside slope, and moderate to severe rocking of 10 to 15 dolosse on the crown and upper lakeside slope was observed throughout Step 2. Dolos displacement was continuing to occur at the end of Step 2 and the step duration was extended 40 min, during which time the damage rate slowed down considerably but did not stop. During Step 3, 10 dolosse were displaced onto the harbor-side slope. Damage had not stopped at the end of the step and it was extended approximately 25 min. By the end of the Step 3 extension, all damage had stopped but continued minor to significant rocking of dolosse was occurring on the upper slope and crown. The structure then was exposed to a fourth hydrograph step which consisted of 25 min of the Step 2 wave condition (8-sec, 15.0-ft nonbreaking wave). This was done to see if damage would be reinitiated at this condition, which appeared to be the worst condition of Hydrograph B relative to the overall stability of the dolos armor. (All three wave conditions of Hydrograph B produced major overtopping.) Seven more dolosse were displaced from the crown onto the harbor-side slope and the test was stopped. It appeared that the damage (dolos displacement) would continue on the upper slope and crown, and the level of damage already exceeded an acceptable amount. Photos 5-7 show the breakwater condition at the end of Hydrographs A and B. During these hydrographs there appeared to be a slow shoreward migration of a few of the existing crown armor stone and several additional dolosse on the crown that were not referred to in the test results reported in this paragraph and paragraph 13.

15. In summary, during Hydrographs A and B, 24 dolosse were displaced from the crown onto the harbor-side slope and 3 dolosse were displaced down the lakeside slope from their original placement location below the swl. Several crown dolosse and crown stone showed some shoreward migration and numerous dolosse around the swl and on the upper lakeside slope and crown exhibited in-place rocking that ranged from minor to major. All wave conditions produced wave overtopping which ranged from moderate to major.

16. The 4-ton dolos armor was removed, the underlayer stone and existing stones on the crown and upper breakwater slopes were restored to their original positions (Photos 8-10), and the dolos armor layers were rebuilt

(Photos 11 and 12). The structure once again was exposed to the wave and swl conditions of Hydrograph A. By the end of the test, the dolos crown showed major damage with 35 dolosse displaced down onto the harbor-side slope and 5 additional crown dolosse had been displaced shoreward but had not moved down onto the harbor-side slope (Photos 13-15). Only one dolos was displaced on the lower lakeside slope. Several dolosse on the upper slope and around the swl and crown exhibited minor to major in-place rocking throughout the test and armor unit displacement did not subside by the end of the test. The hydrograph was not extended since it was thought that damage (dolos displacement) already had exceeded a desirable level. Due to the extensive damage accrued during Hydrograph A it was decided that there was no need to expose the section to Hydrograph B.

17. Prior to the second testing of the 4-ton dolos rehabilitation section NCB had decided, based on the first testing with Hydrograph A, that the 4-ton dolosse was an inadequate design for the 13.4-ft nonbreaking wave condition. NCB requested that at conclusion of repeat testing of 4-ton dolosse, WES initiate a test series to determine the actual design level wave height for which the 4-ton dolos showed acceptable stability.

18. It was noted during the second testing of Plan 1 with Hydrograph A that some of the dolos displacement occurred during impact of the last wave in the wave train produced during a 30-sec test cycle. The wave is typically of a longer period than the rest of the waves due to wave generator effects. To eliminate the possibility of this wave having an influence on the test results, the last wave in the wave train was filtered out in all subsequent tests.

19. The 4-ton dolos armor was rebuilt and Plan 1 was exposed to 8.0-sec nonbreaking wave heights of 11.0, 11.5, 12.0, and 12.5 ft at an swl of +4.9 ft lwd. The 12.5-ft wave produced damage (dolos displacement) that exceeded an acceptable amount on the breakwater crown. Wave heights below this had caused only minor to moderate damage. The 4-ton dolos armor layers were rebuilt (Photo 16) and the structure was exposed to the wave and swl conditions of Hydrograph C (Table 3). Two dolosse were displaced from the crown onto the harbor-side slope during Step 1. Step 2 caused no dolosse displacement, but two additional crown dolosse were displaced onto the harbor-side slope during the early part of Step 3. All damage had stopped during the last 30 min of Step 3. Steps 1 and 2 caused only minor rocking of a few dolosse at the swl

and upper lakeside slope. Step 3 produced a slight increase in the severity of in-place dolos rocking, but this was still only moderate in its severity. Plan 1 was in good condition with only minor damage at the end of Hydrograph C (Photos 17-19). All of the displaced dolosse had come from the harbor side of the crown where, due to the random placement, some of the dolosse were not interlocked with other dolos armor.

20. Without rebuilding the dolos armor, the structure was exposed to Hydrograph A. No additional dolos displacement occurred during Hydrograph A, but the amount and severity of in-place dolos rocking showed a definite increase on the crown and upper lakeside slope. The structure was in good condition at the end of Hydrograph A (Photos 20-22), but the moderate to significant in-place rocking observed during this test showed that there was a high potential for possible dolos displacement and breakage.

Plan 2, 2.3-ton Dolosse

21. With the proposed 4-ton dolos design (Plan 1, Plate 1) proven to be stable for the wave and swl conditions of Hydrograph C, NCB requested that additional tests be initiated to determine the stable design wave height for the existing 2-ton dolosse (Plan 2, Plate 2) on the east breakwater at Cleveland Harbor.

22. The existing 2-ton dolosse on the east breakwater in Cleveland Harbor could not be represented exactly in the model without changing the existing model scale and recalibrating the test facility. At the existing scale and with the available model dolosse, a 2.3-ton dolos with a specific weight of 143 pcf could be represented. NCB thought this representation would be the most cost-effective approach and results of these tests would provide the information they required.

23. The 2.3-ton dolos section (Plan 2) had a crown width of 13.0 ft at an elevation of +10.3 ft lwd. Two layers of randomly placed dolosse extended down to an elevation of -22.0 ft lwd on a 1V-on-2H slope. Plan 2 was exposed to 8-sec nonbreaking wave heights of 8.5, 9.0, 10.0, 10.5, and 11.0 ft at an swl of +4.9 ft lwd. Although no significant dolos displacement occurred, the 11.0-ft wave height produced significant amounts and degrees of in-place rocking of the dolosse on the crown and upper lakeside slope. Wave heights below this had caused only minor displacement and in-place rocking. The 2.3-ton

dolos armor layers were rebuilt using totally random placement (Photos 23-25), and the structure was exposed to the wave and swl conditions of Hydrograph D (Table 4). Due to the limited number of model dolosse which reproduced the 2.3-ton dolosse, the outer 1-ft sections of the lakeside slope adjacent to the flume walls were constructed using a larger size of dolos unit. Therefore these areas were ignored and only the center 4.75-ft width of the test section was observed and reported on for the testing of Plan 2. The dolosse exhibited only slight in-place rocking on the crown and upper lakeside slope during Steps 1 and 2 of Hydrograph D. During Step 3, two dolosse were displaced from the crown onto the harbor-side slope; a few crown dolosse showed a slight shift toward the harbor side and minor rocking was observed on the crown and upper slope. All damage occurred early in Step 3 and the structure was in good condition (slight spot damage to dolosse) at the end of the test (Photos 26-28).

24. The 2.3-ton dolos armor layers were rebuilt and the structure was exposed once again to Hydrograph D. Results of this test were very similar to the first testing with the observance of slight to minor rocking of a few dolosse on the crown and upper lakeside slope throughout the entire test. During Step 3, two dolosse were displaced from the crown onto the harbor-side slope and several of the crown dolosse showed a slight shift toward the harbor side of the test section. The damage had subsided and the structure showed only slight spot damage to the dolosse at the end of the test (Photos 29-31).

Plan 3, 9- to 20-ton Armor Stone

25. With the completion of tests for Plan 2, tests were initiated to determine the design wave condition for the proposed 9- to 20-ton armor-stone rehabilitation design alternative (Plan 3, Plate 3). The model armor stone represented a uniformly distributed gradation of 9- to 20-ton armor stone randomly placed on a 1V-on-1.5H slope. The armor stone had a crown width of 26 ft at an elevation of +10.3 ft lwd and extended down to the -22.0 ft lwd toe berm.

26. After approximately 15 min of 8.0-sec, 7.0-ft shakedown waves, Plan 3 was exposed to 8.0- and 9.0-sec, 12.5-, 13.0-, and 13.4-ft nonbreaking waves. Wave heights below 13.4 ft caused only minor rocking and very slight in-place reorientation of a few armor stones on the crown and upper lakeside

slope, while the 13.4-ft waves caused some armor-stone displacement in these same areas. Based on these observations, Plan 3 (Photos 32-34) was exposed to the wave and swl conditions of Hydrograph A. During Step 1, two stones at the swl were displaced downslope and one crown stone was displaced down the harbor-side slope. Two additional stones were displaced downslope from the swl during Step 2. Five additional armor stones (one on the crown and four at the swl) showed moderate to significant in-place rocking throughout the test. No armor-stone displacement occurred during Step 3 and the structure was in good condition (slight to minor spot damage on crown and around swl) at the end of the test (Photos 35-37). The displaced armor stone and stone exhibiting in-place rocking during this test ranged in weight from the smallest to largest in the graded armor-stone mix.

27. The 9- to 20-ton armor stone was rebuilt (Photos 38-40) and Plan 3 was once again exposed to Hydrograph A. The structure accrued less damage during this testing. Three armor stones were displaced (two on the crown and one at the swl on the lakeside slope) during Steps 1 and 2. No displacement occurred during Step 3, but two armor stones on the crown and three at the swl continued to show minor to significant in-place rocking and reorientation throughout the test. Photos 41-43 show that the structure was in good condition (slight spot damage on crown and around swl) at the end of the test.

PART IV: CONCLUSIONS

28. Based on the test conditions and test results reported herein, it is concluded that:

a. Plan 1 (4-ton, 140-pcf dolosse) might prove capable of withstanding the wave and swl conditions of Hydrograph A, but the structure shows a high potential to sustain significant damage on the crown and upper lakeside slope. Thus it is thought to be a very marginally acceptable design that probably will require significant amounts of maintenance if exposed to conditions similar to those of Hydrograph A (Table 1).

b. Plan 1 is a very inadequate design for wave and swl conditions of Hydrograph B (Table 2) and could accrue extensive damage to the dolosse on the crown and upper lakeside slope if exposed to these conditions for any length of time.

c. Plan 1 appears to be a stable design (sustaining only minor crown damage) for the wave and swl condition of Hydrograph C (Table 3).

d. Plan 2 (2.3-ton, 143-pcf dolosse) appears to be a stable design and should sustain only minor crown damage when exposed to the wave and swl conditions of Hydrograph D (Table 4).

e. Plan 3 (9- to 20-ton, 155-pcf armor stone) appears to be a stable design and should sustain only slight to minor damage in an area extending from the swl to the crown when exposed to the wave and swl conditions of Hydrograph A.

PART V: DISCUSSION

29. During construction and testing of the 2.3- and 4.0-ton dolos designs it was observed that stability of the crown dolosse tended to vary across the width of the test section. The dolos construction was totally random and tapered out to a one-layer placement on the harbor side of the crown. It was noted that if dolosse in this one-layer area were either interlocked with an adjacent dolos unit or if a fluke of the dolos projected down into a void area between the armor stone, the dolos exhibited a higher stability than those which ended up in a solitary position on a flat portion of the crown. Based on these observations, it would appear that dolosse units placed in this one-layer area of the crown are subject to easy displacement by overtopping waves and they need to be keyed into the existing armor stone or adjacent dolosse units. In areas where this keying of the one-layer dolosse cannot be achieved, it may be a better alternative to not place a dolos unit, as it will most likely be displaced during the first overtopping storm condition that occurs.

30. The 9- to 20-ton armor-stone design exhibited the highest stability of the plans tested, but regardless of whether it or the 4-ton dolos design is selected for the proposed rehabilitation work, care must be taken to tie the new protection into the existing keyed-and-fitted armor stone on the crown and ends of the rehabilitation areas and the 2.0-ton dolosse in the eastern limits of the new work. If a straight-line transition is used between either the 4-ton dolosse or 9- to 20-ton armor stone and the existing 2.0-ton dolosse, this area could prove to be an area of inherent instability and may require continual maintenance after storm-wave conditions. Efforts should be made to interlock this area of dissimilar armor. The end areas of the new rehabilitation also could be subject to damage, especially for storm waves that approach this area from an oblique angle. These areas should be constructed so that a smooth transition exists from the new work into the existing breakwater armor. If this cannot be achieved, large buttressing stone could be placed from the toe to the crown on the ends of the new rehabilitation work to prevent the displacement of new dolos or armor stone when storm waves approach the structure from oblique angles.

Table 1

Hydrograph A

Step	swl ft lwd	Test Wave Period sec	Test Wave Height* ft	Prototype** Duration min	Wave Type
	+4.9	7.0	7.0	15.0	Shakedown
1	+4.9	7.0	13.4	40.0	Nonbreaking
2	+4.9	8.0	13.4	40.0	Nonbreaking
3	+4.9	9.0	13.4	40.0	Nonbreaking

* Wave height measured in a water depth of 34.9 ft.
** Test durations varied in some instances as designated in text of report.

Table 2

Hydrograph B

Step	swl ft lwd	Test Wave Period sec	Test Wave Height* ft	Prototype** Duration min	Wave Type
1	+4.9	7.0	15.0	40.0	Nonbreaking
2	+4.9	8.0	15.0	40.0	Nonbreaking
3	+4.9	9.0	15.0	40.0	Nonbreaking

* Wave height measured in a water depth of 34.9 ft.
** Test durations varied in some instances as designated in text of report.

Table 3

Hydrograph C

Step	swl ft lwd	Test Wave Period sec	Test Wave Height* ft	Prototype Duration min	Wave Type
	+4.9	7.0	7.0	15.0	Shakedown
1	+4.9	7.0	12.0	40.0	Nonbreaking
2	+4.9	8.0	12.0	40.0	Nonbreaking
3	+4.9	9.0	12.0	40.0	Nonbreaking

* Wave height measured in a water depth of 34.9 ft.

Table 4

Hydrograph D

Step	swl ft lwd	Test Wave Period sec	Test Wave Height* ft	Prototype Duration min	Wave Type
	+4.9	7.0	6.0	15.0	Shakedown
1	+4.9	7.0	10.5	40.0	Nonbreaking
2	+4.9	8.0	10.5	40.0	Nonbreaking
3	+4.9	9.0	10.5	40.0	Nonbreaking

* Wave height measured in a water depth of 34.9 ft.

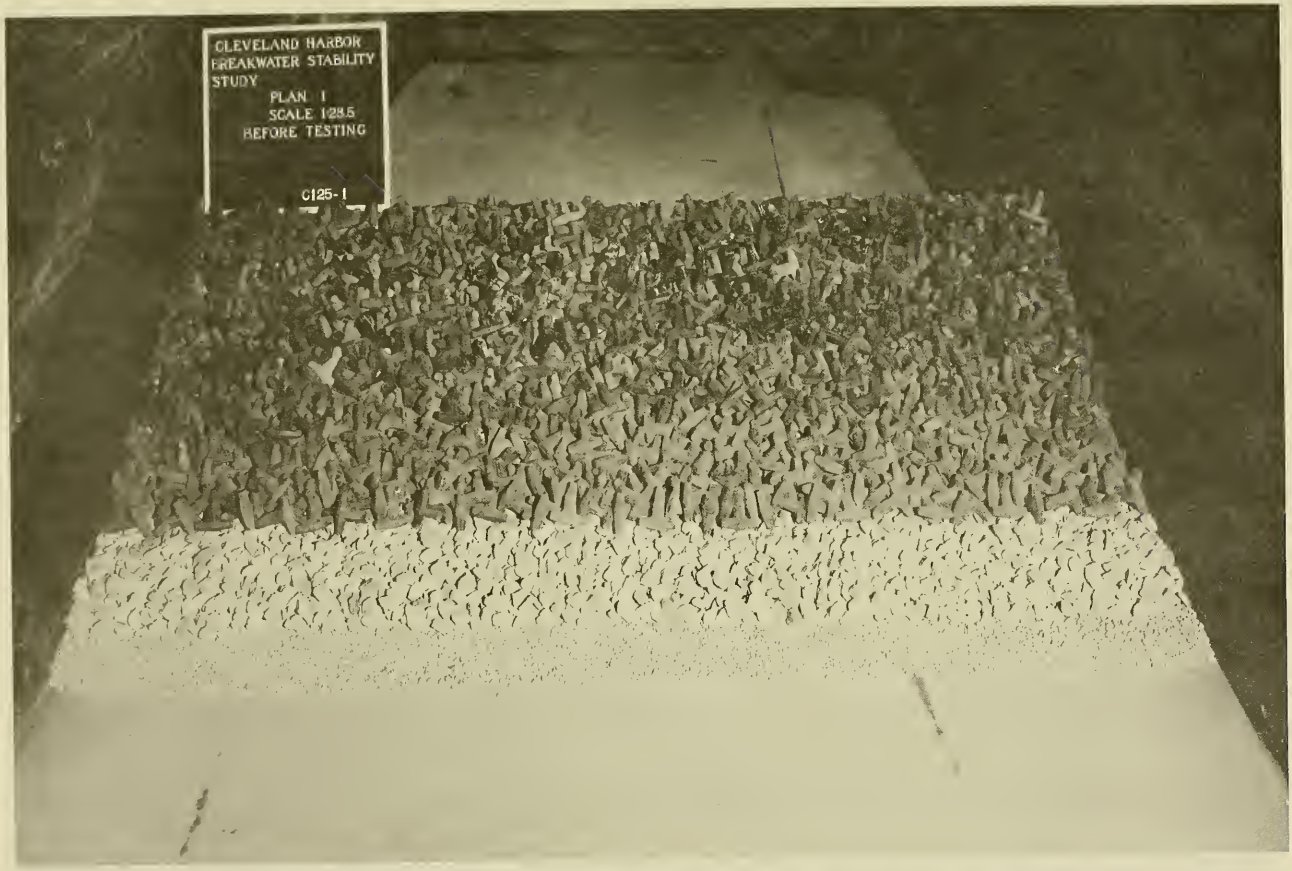


Photo 1. Lakeside view of Plan 1 before testing, first test section

Photo 2. Harbor-side view of Plan 1 before testing, first test section

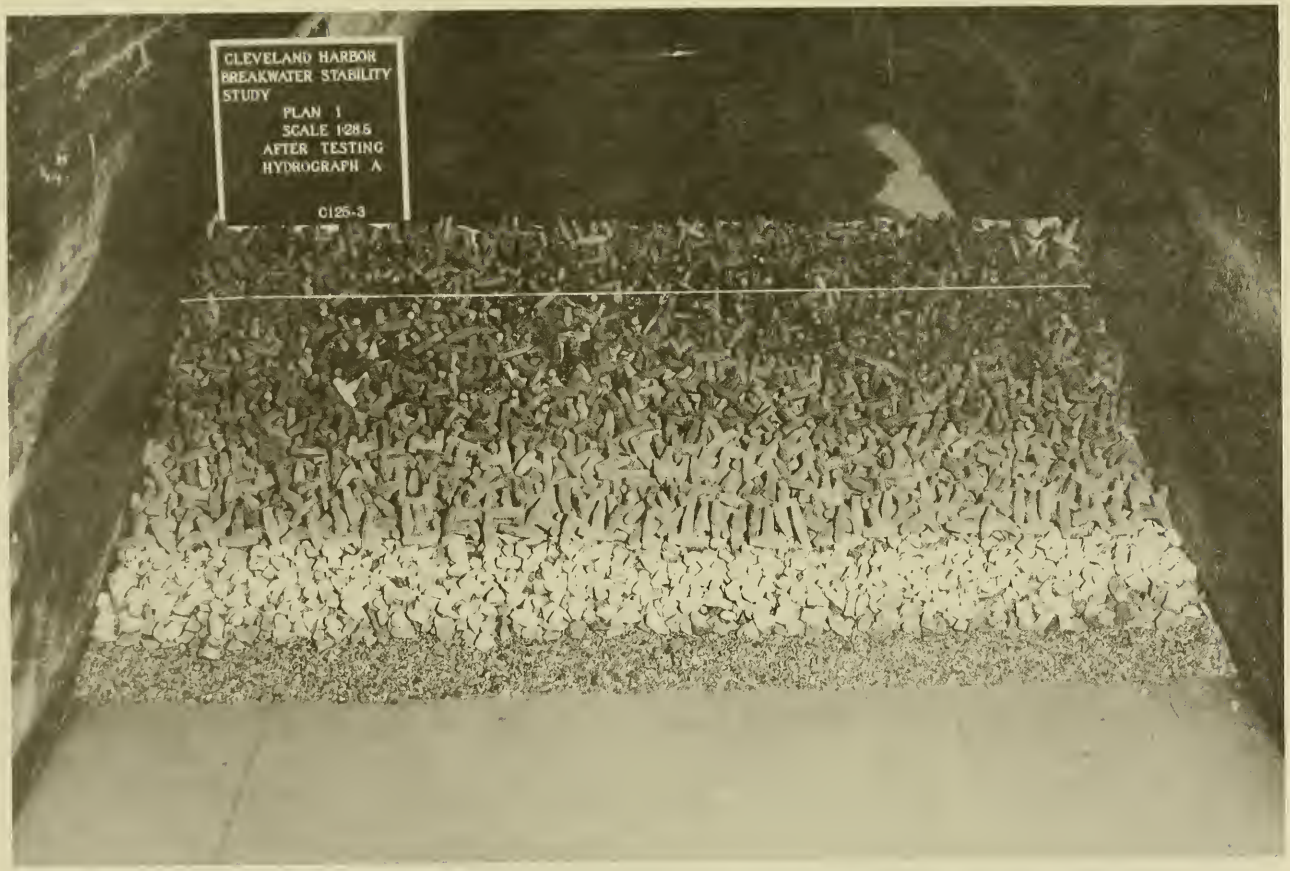


Photo 3. Lakeside view of Plan 1 after testing Hydrograph A, first test section

Photo 4. Harbor-side view of Plan 1 after testing Hydrograph A, first test section

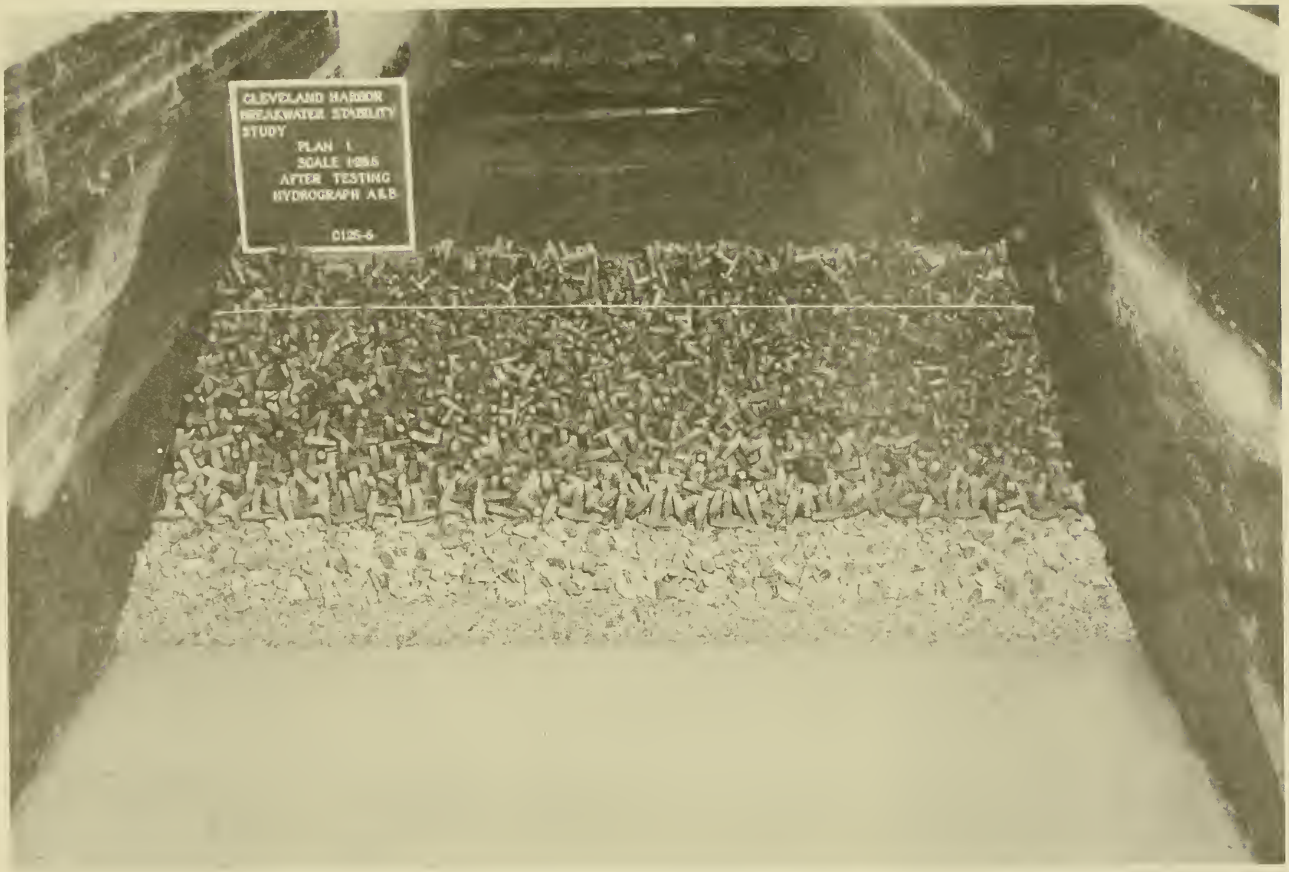


Photo 5. Lakeside view of Plan 1 after testing Hydrographs A and B, first test section

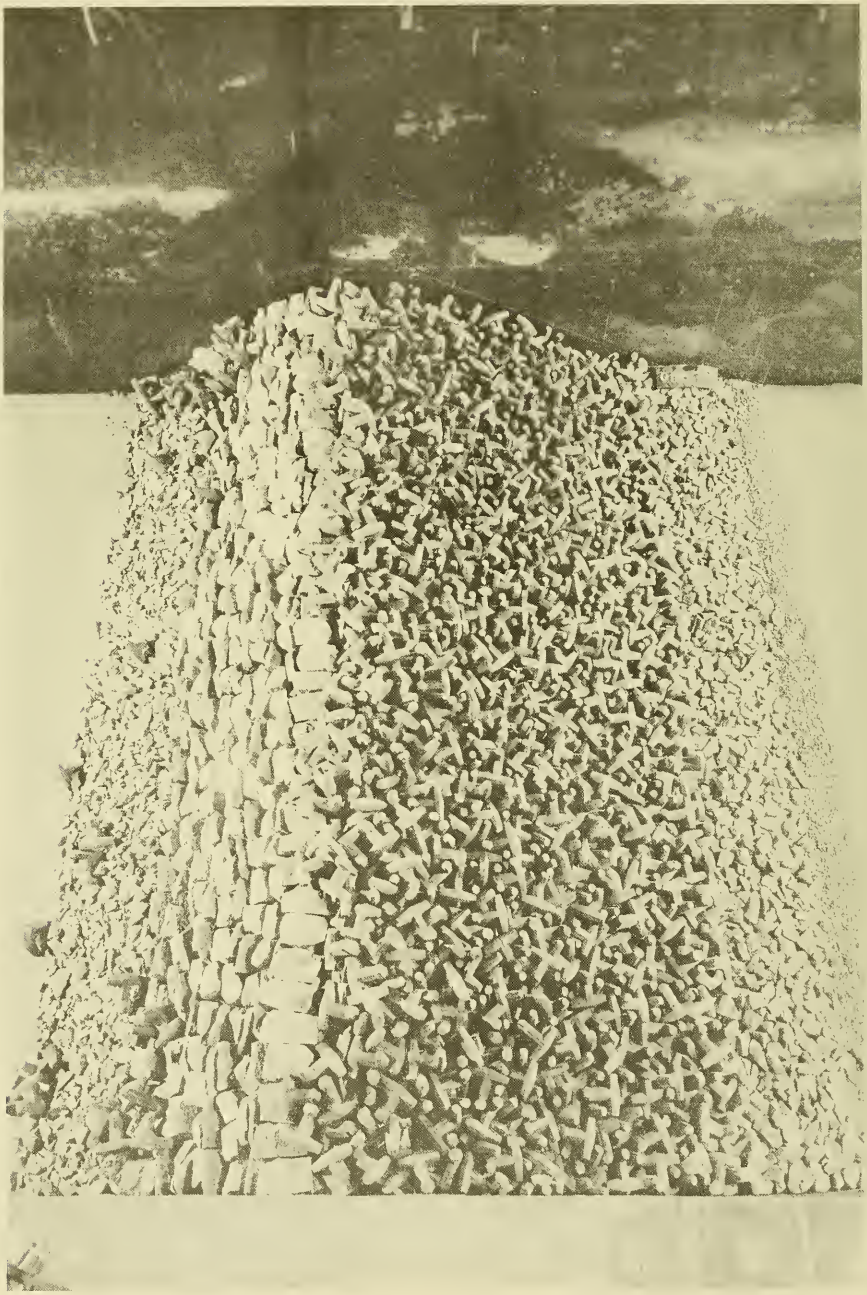


Photo 6. Overhead view of Plan 1 after testing Hydrographs A and B, first test section

Photo 7. Harbor-side view of Plan 1 after testing Hydrographs A and B, first test section

Photo 8. Lakeside view of Plan 1 prior to placement of dolos armor, second test section

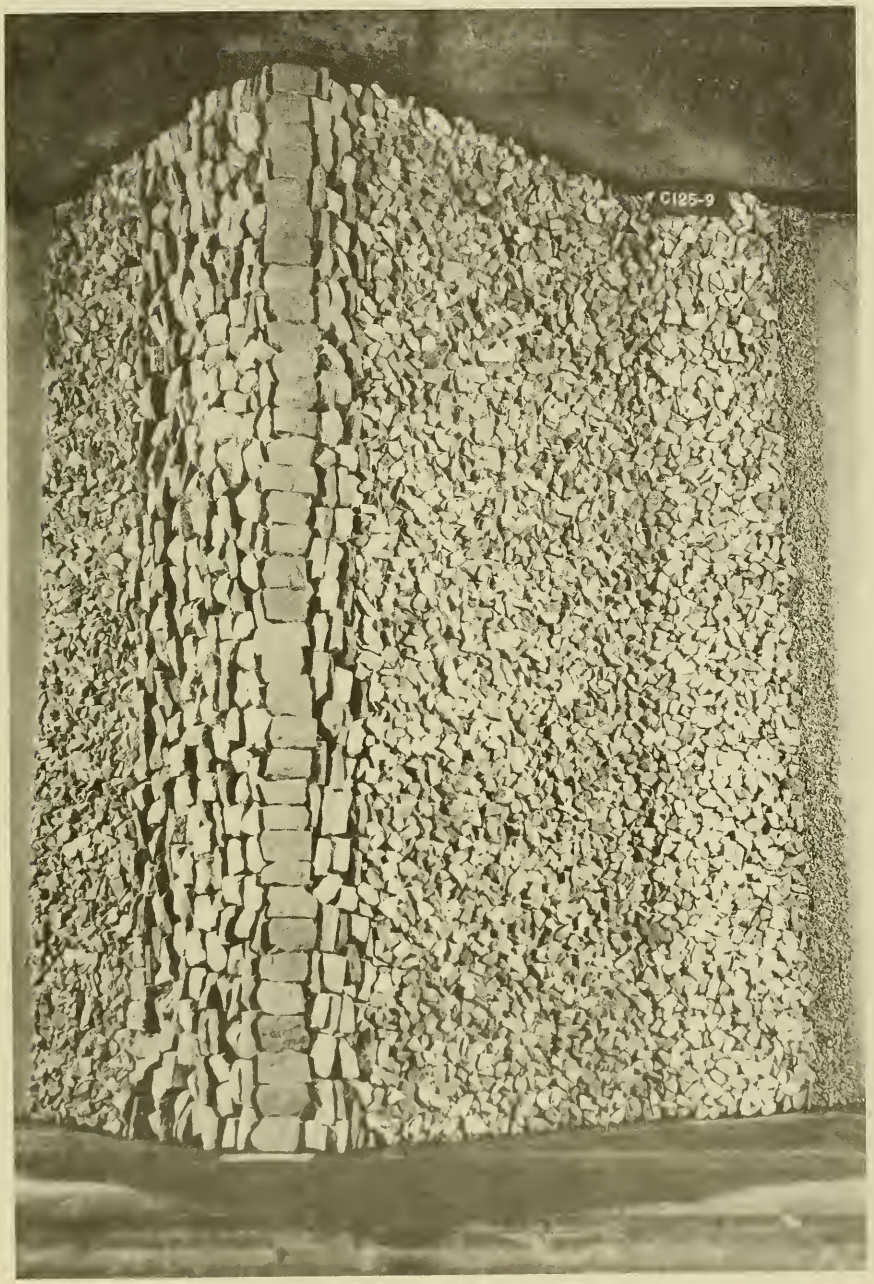


Photo 9. Overhead view of Plan 1 prior to placement of dolos armor, second test section

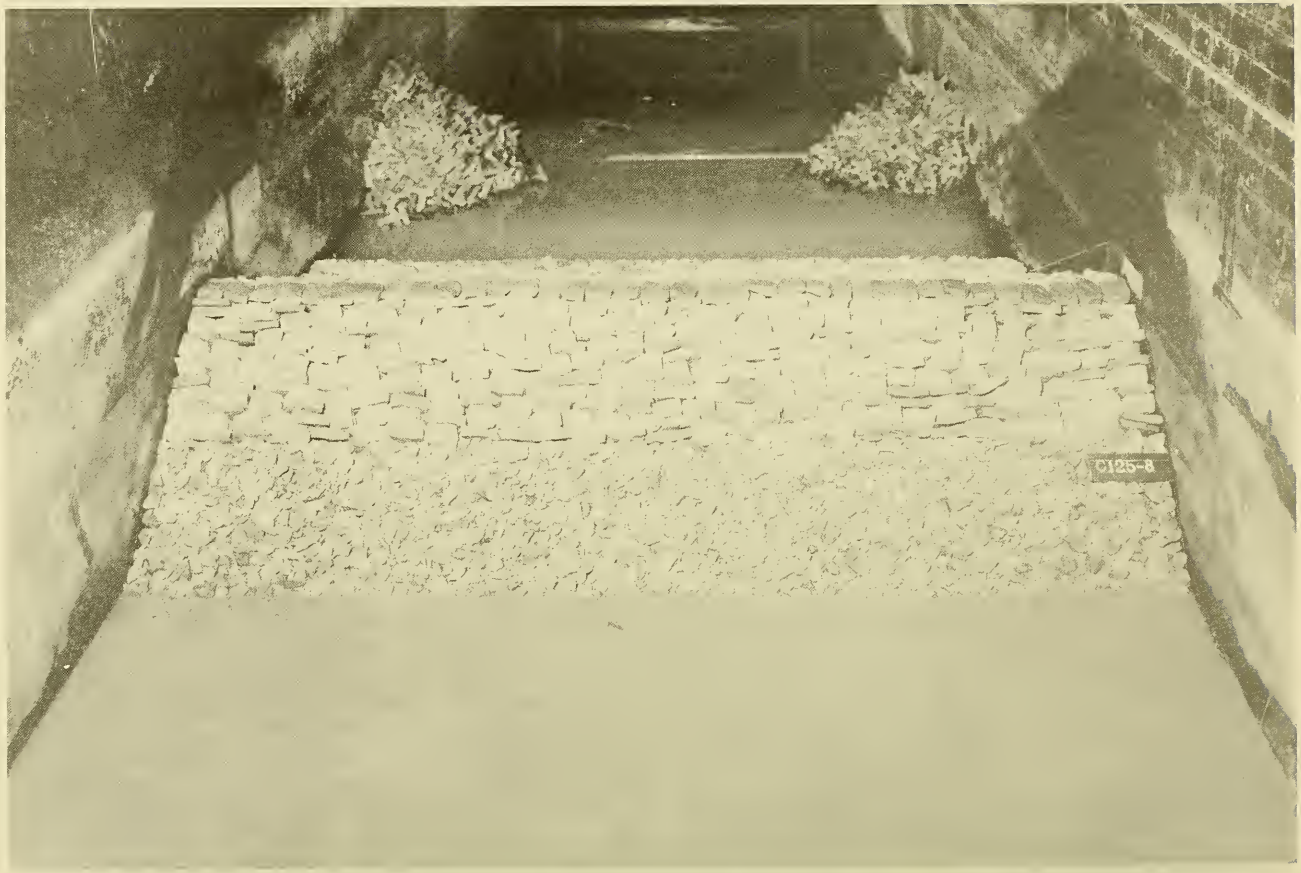


Photo 10. Harbor-side view of Plan 1 prior to placement of dolos armor, second test section

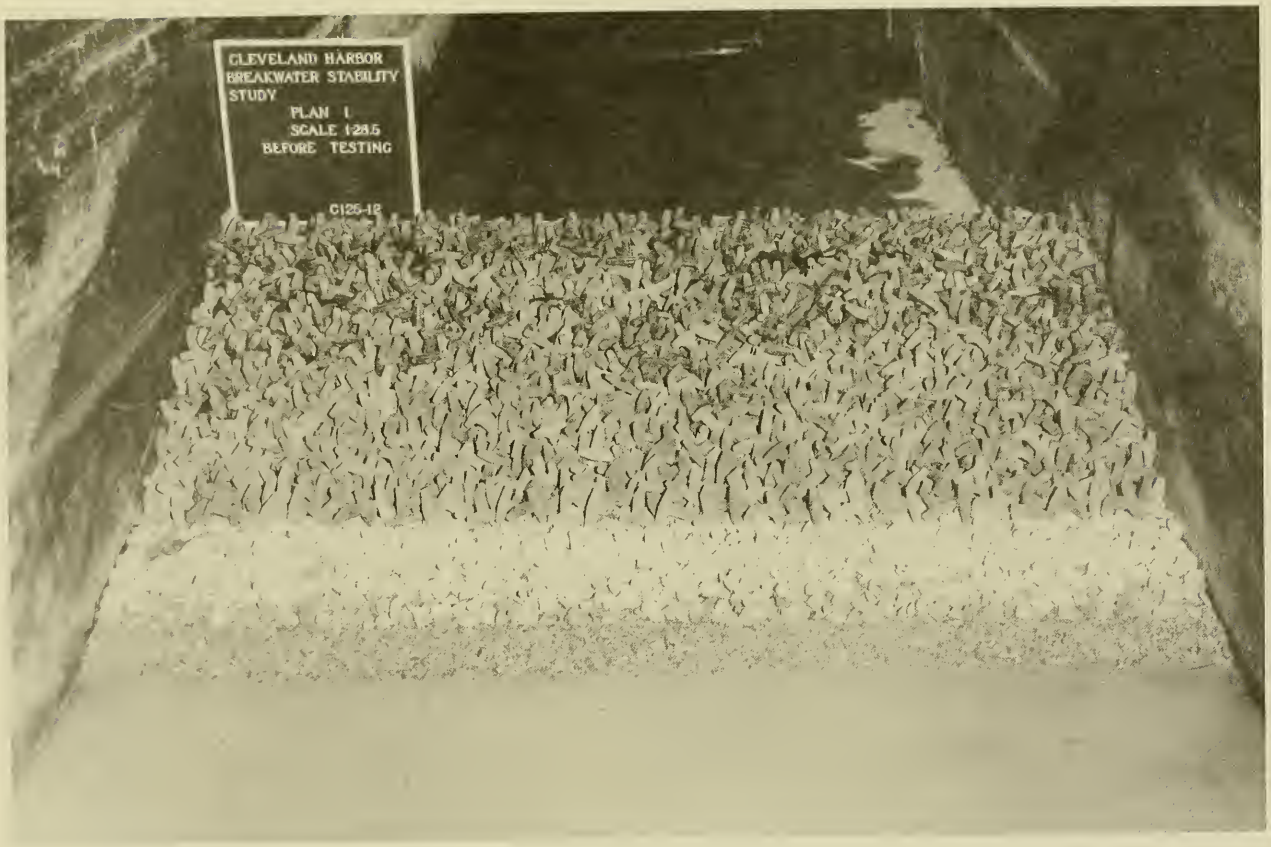


Photo 11. Lakeside view of Plan 1 before testing, second test section

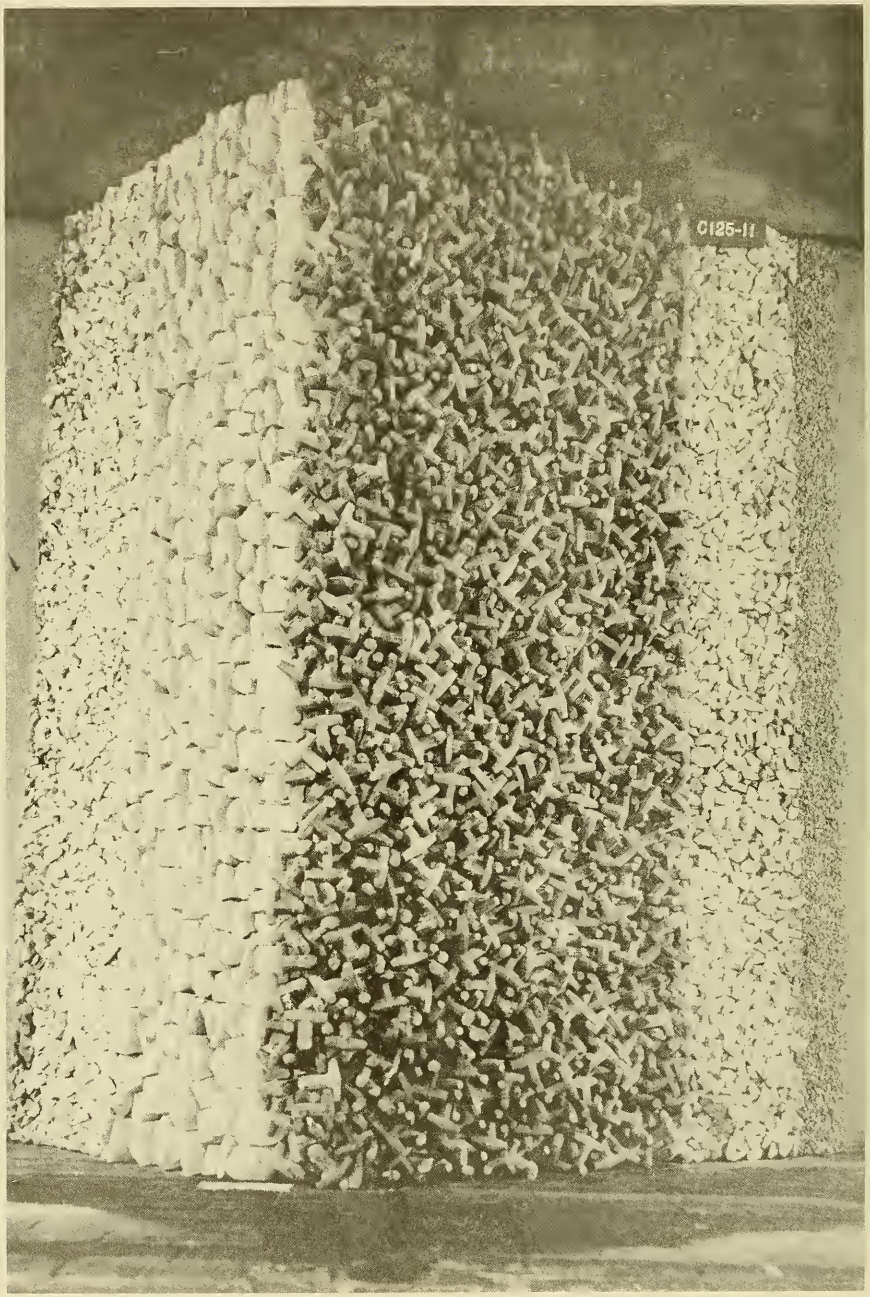


Photo 12. Overhead view of Plan 1 before testing, second test section

Photo 13. Lakeside view of Plan 1 after testing Hydrograph A, second test section

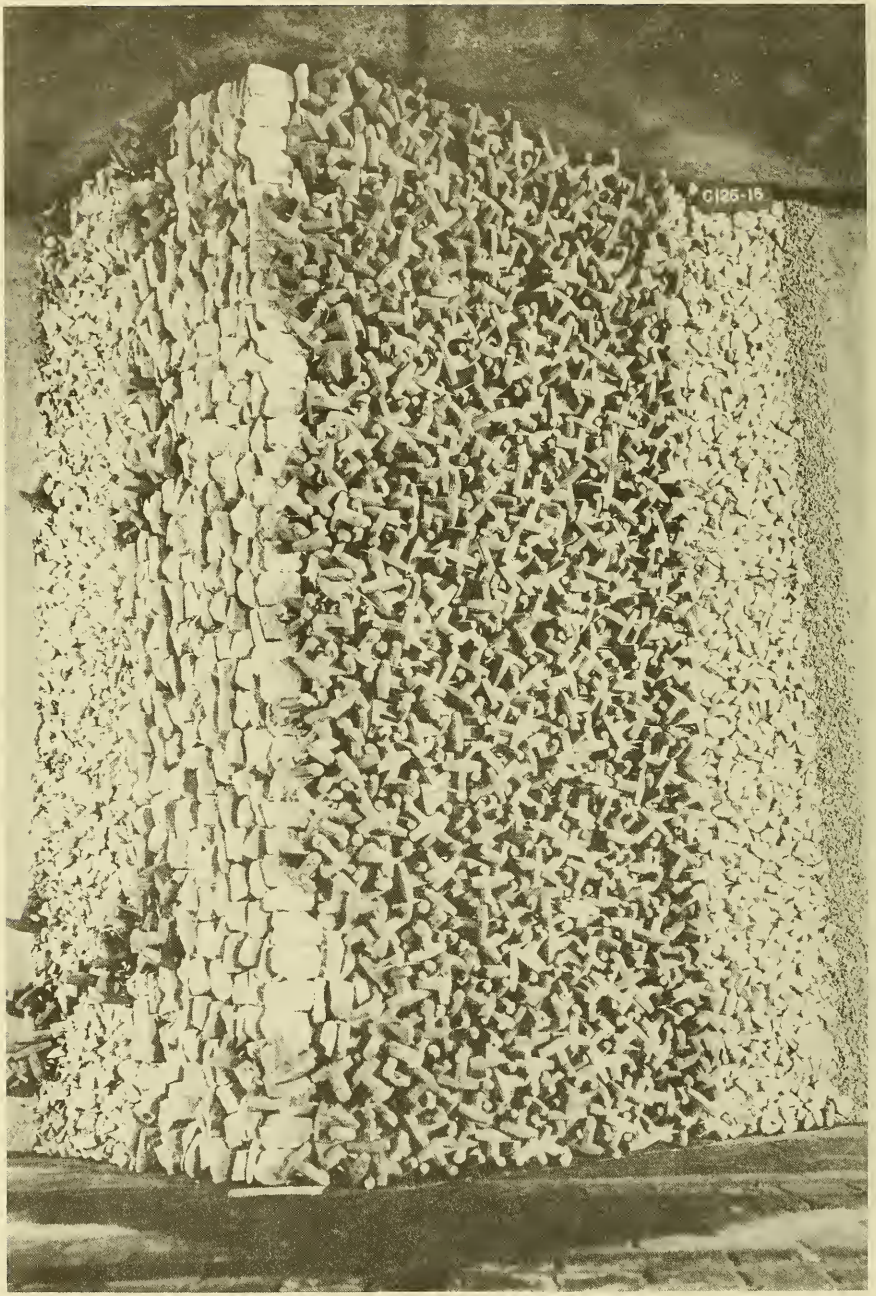


Photo 14. Overhead view of Plan 1 after testing Hydrograph A, second test section

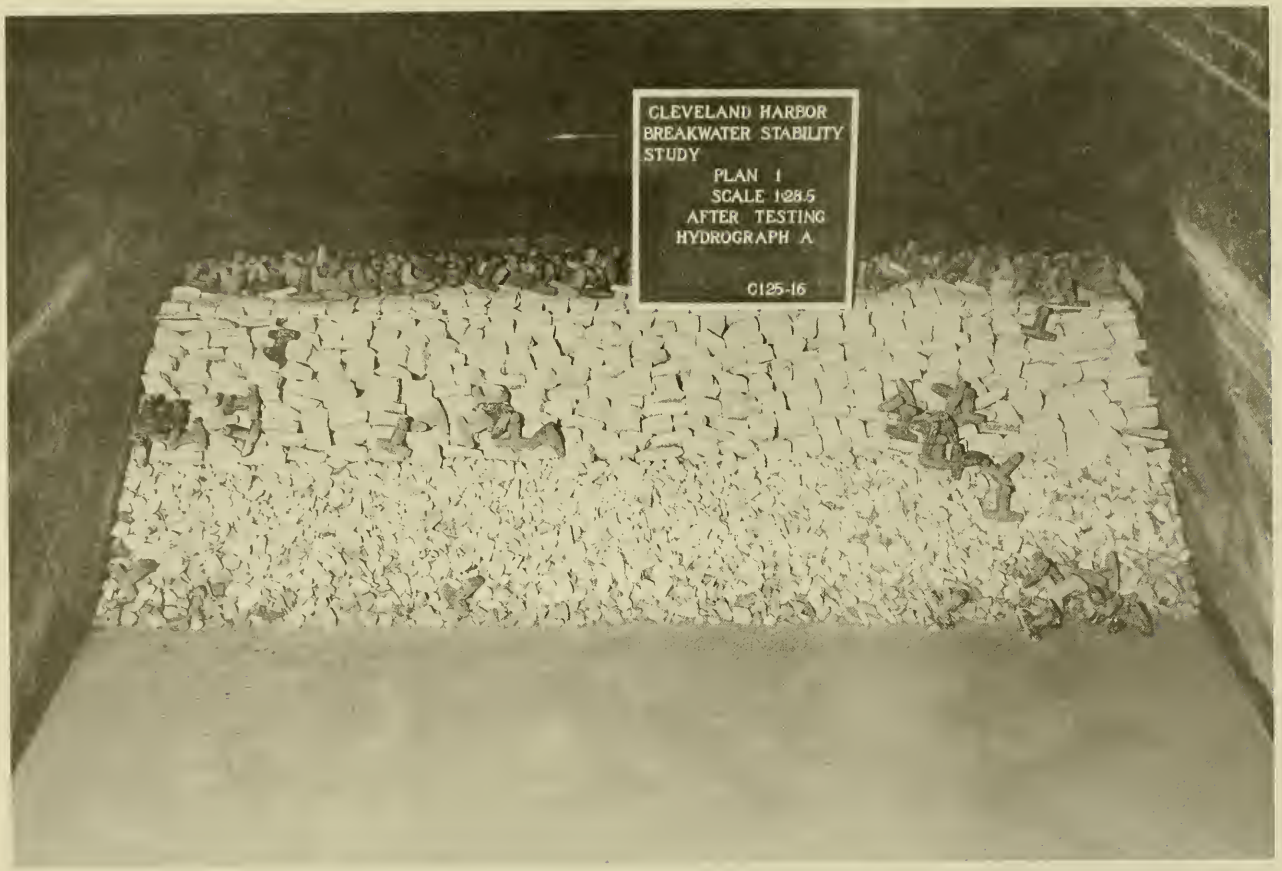


Photo 15. Harbor-side view of Plan 1 after testing Hydrograph A, second test section

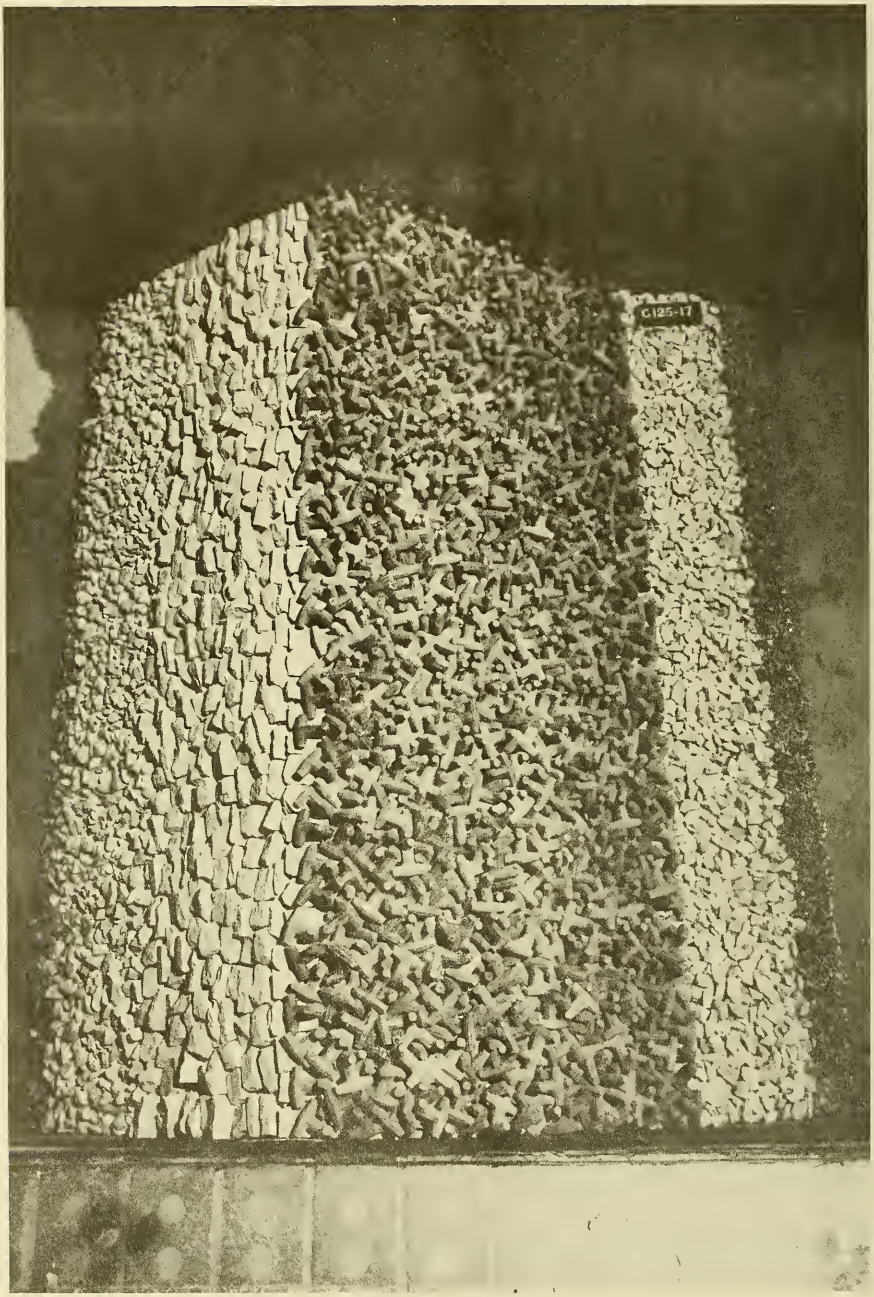


Photo 16. Overhead view of Plan 1 before testing, third test section

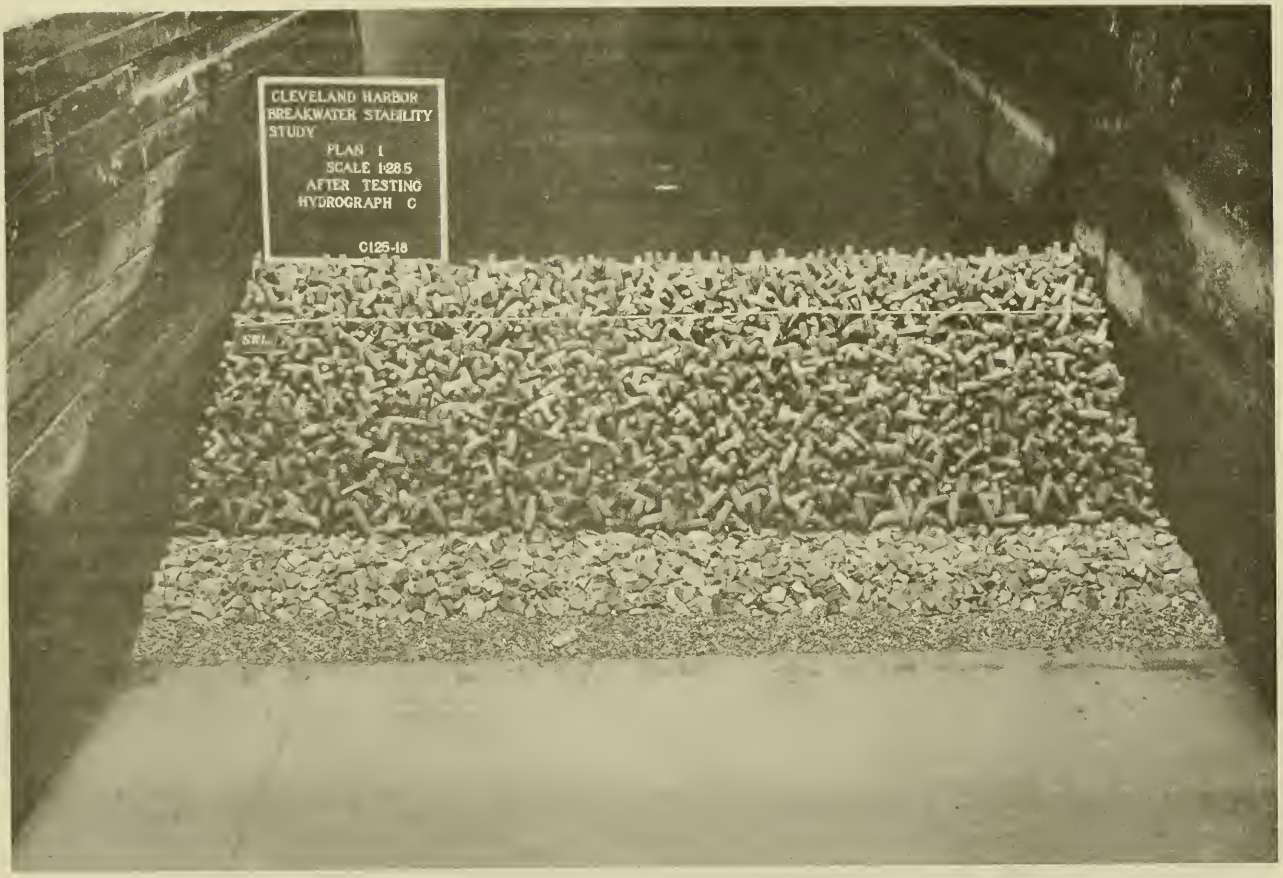


Photo 17. Lakeside view of Plan 1 after testing Hydrograph C, third test section

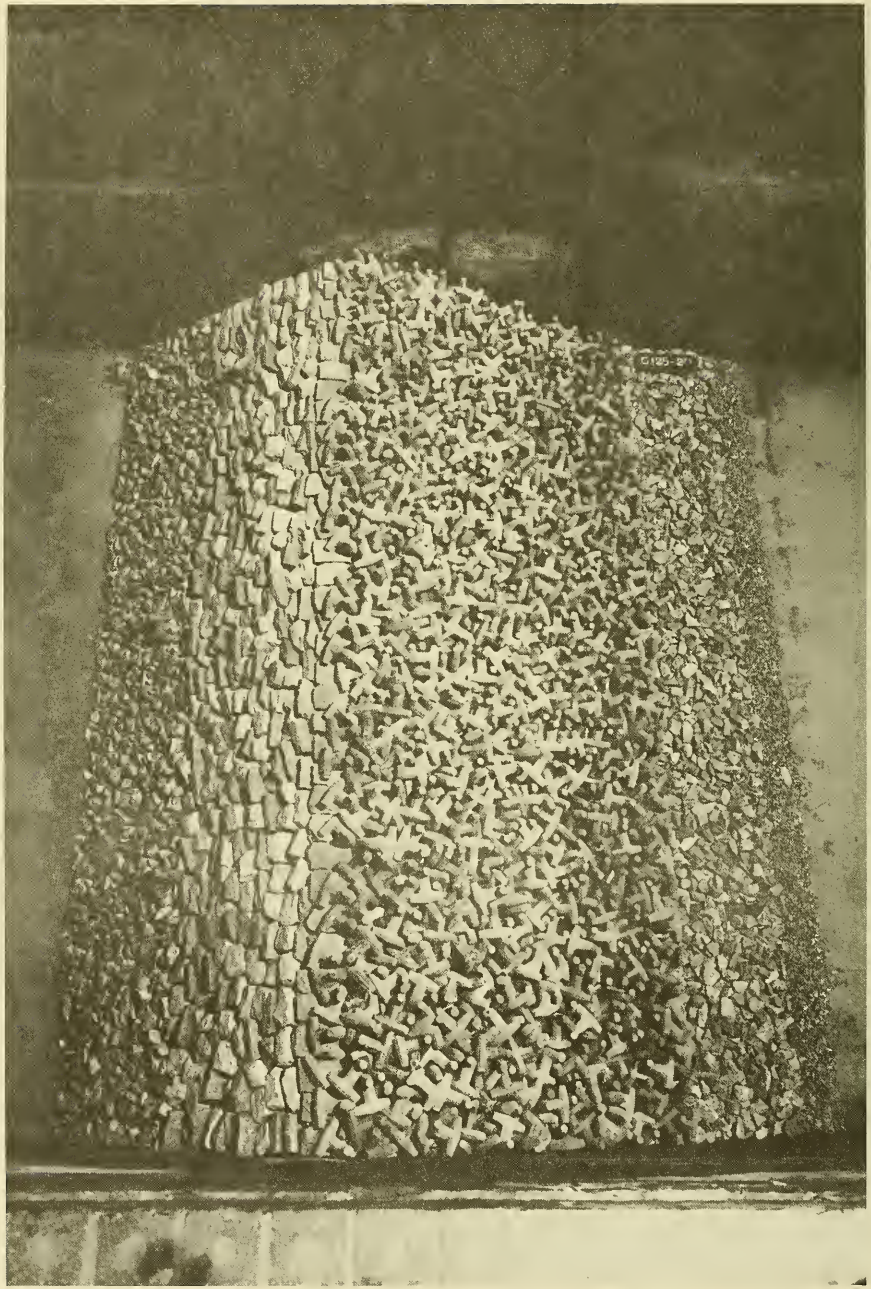


Photo 18. Overhead view of Plan 1 after testing Hydrograph C, third test section

Photo 19. Harbor-side view of Plan 1 after testing Hydrograph C, third test section

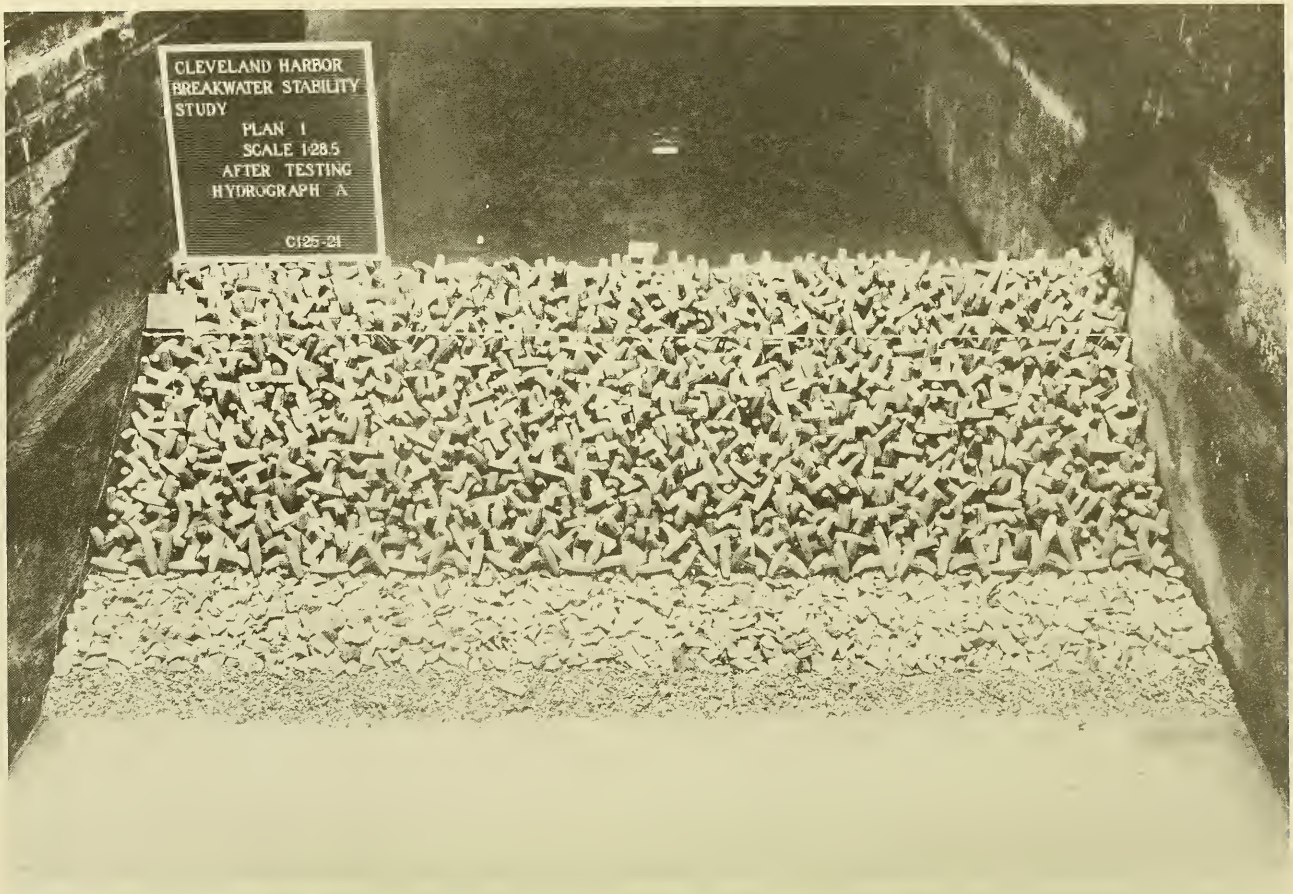


Photo 20. Lakeside view of Plan 1 after testing Hydrographs C and A, third test section

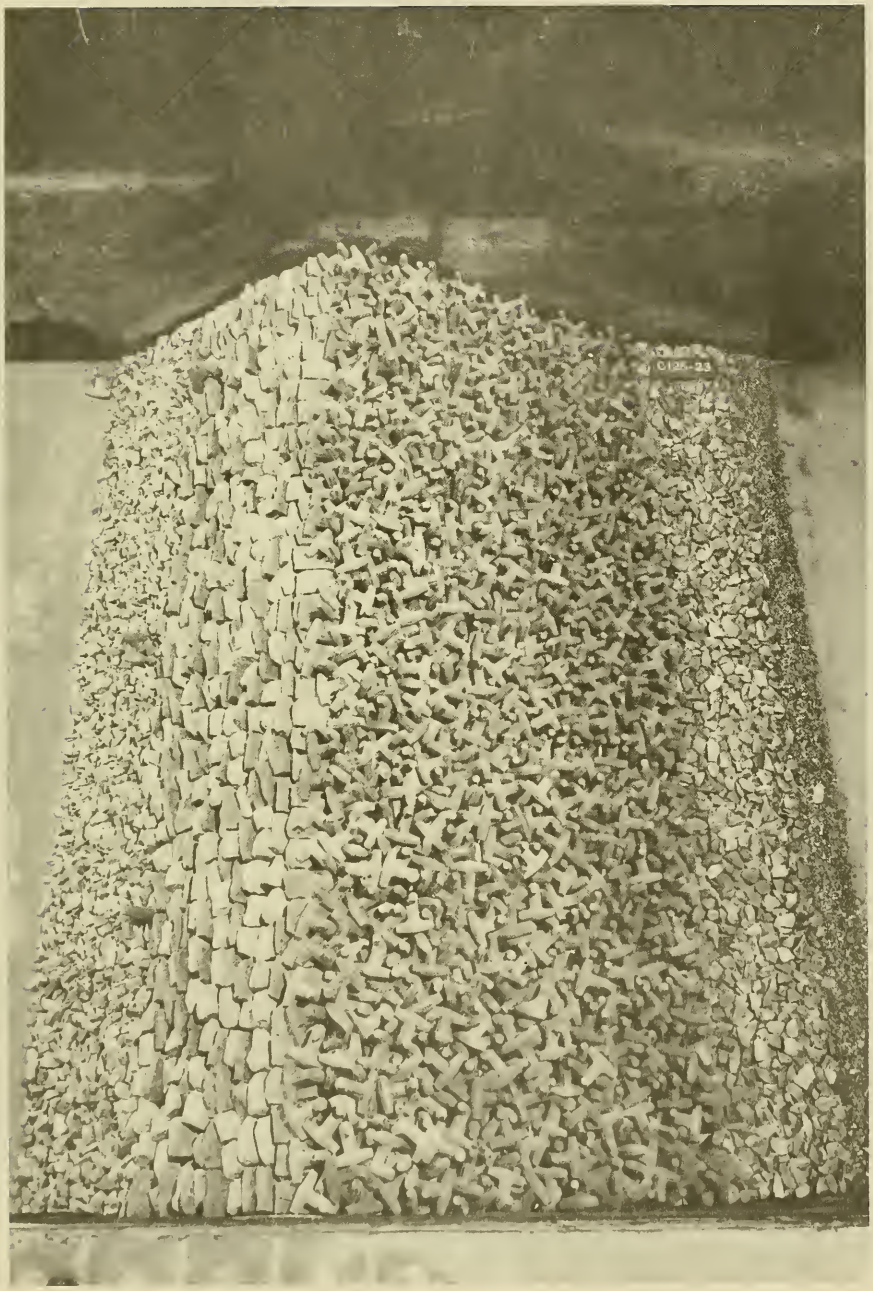


Photo 21. Overhead view of Plan 1 after testing Hydrographs C and A, third test section

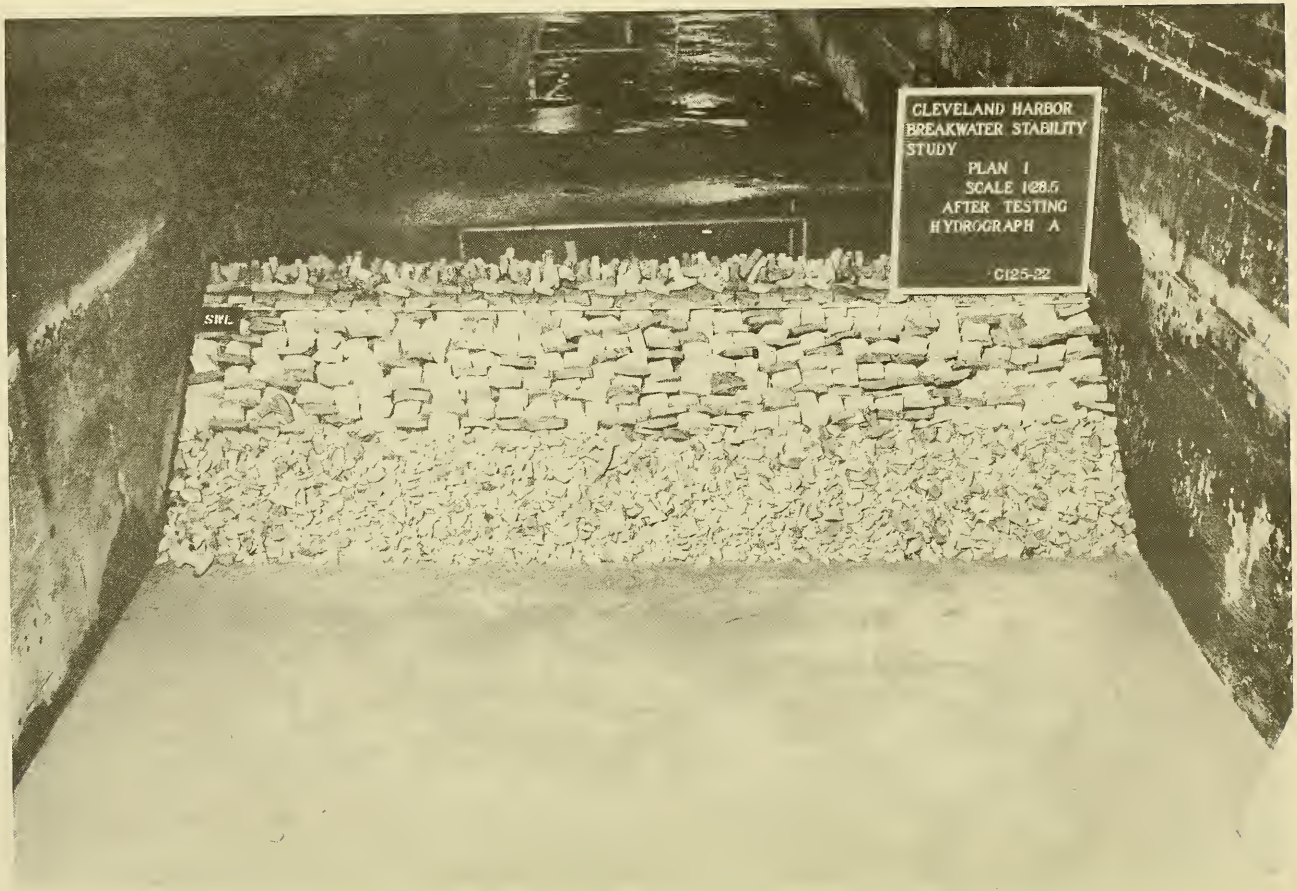


Photo 22. Harbor-side view of Plan 1 after testing Hydrographs C and A, third test section

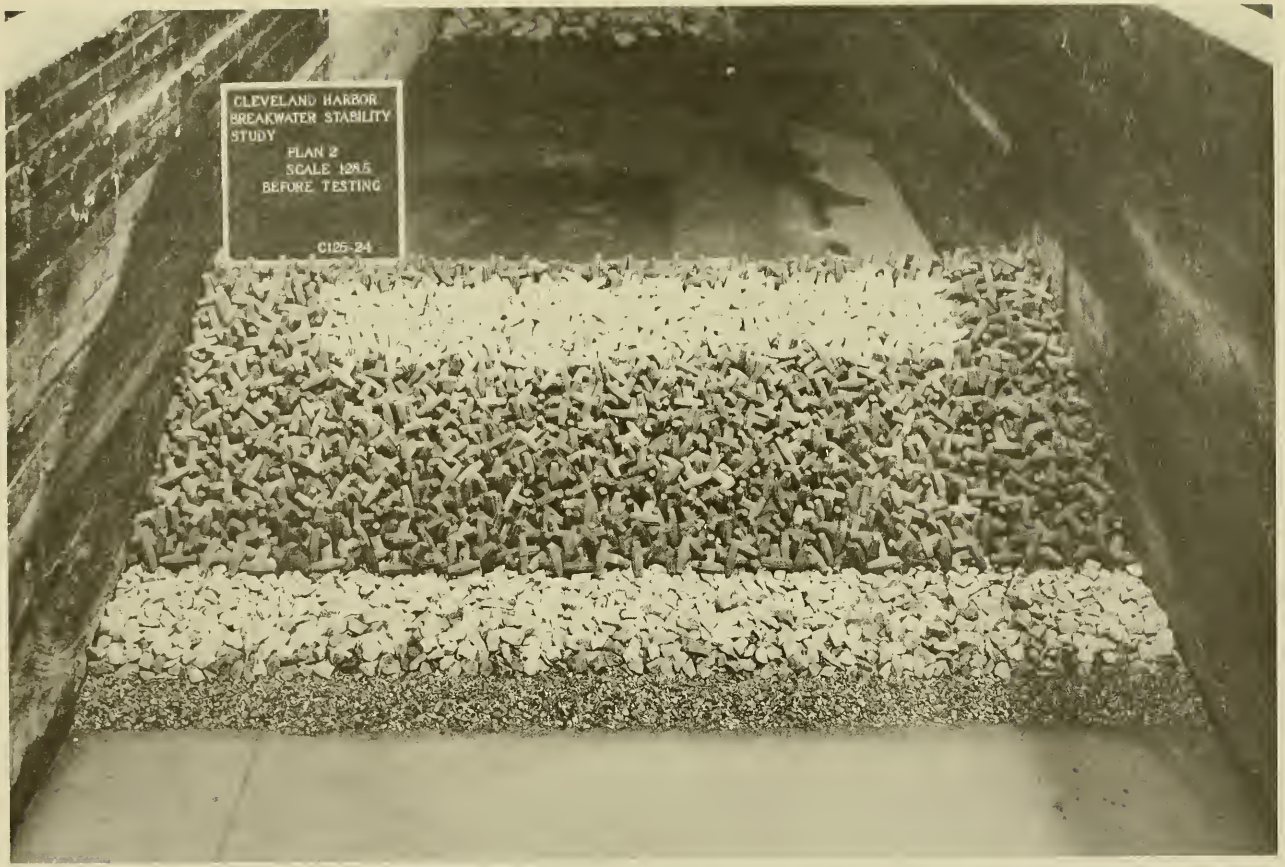


Photo 23. Lakeside view of Plan 2 before testing, first test section

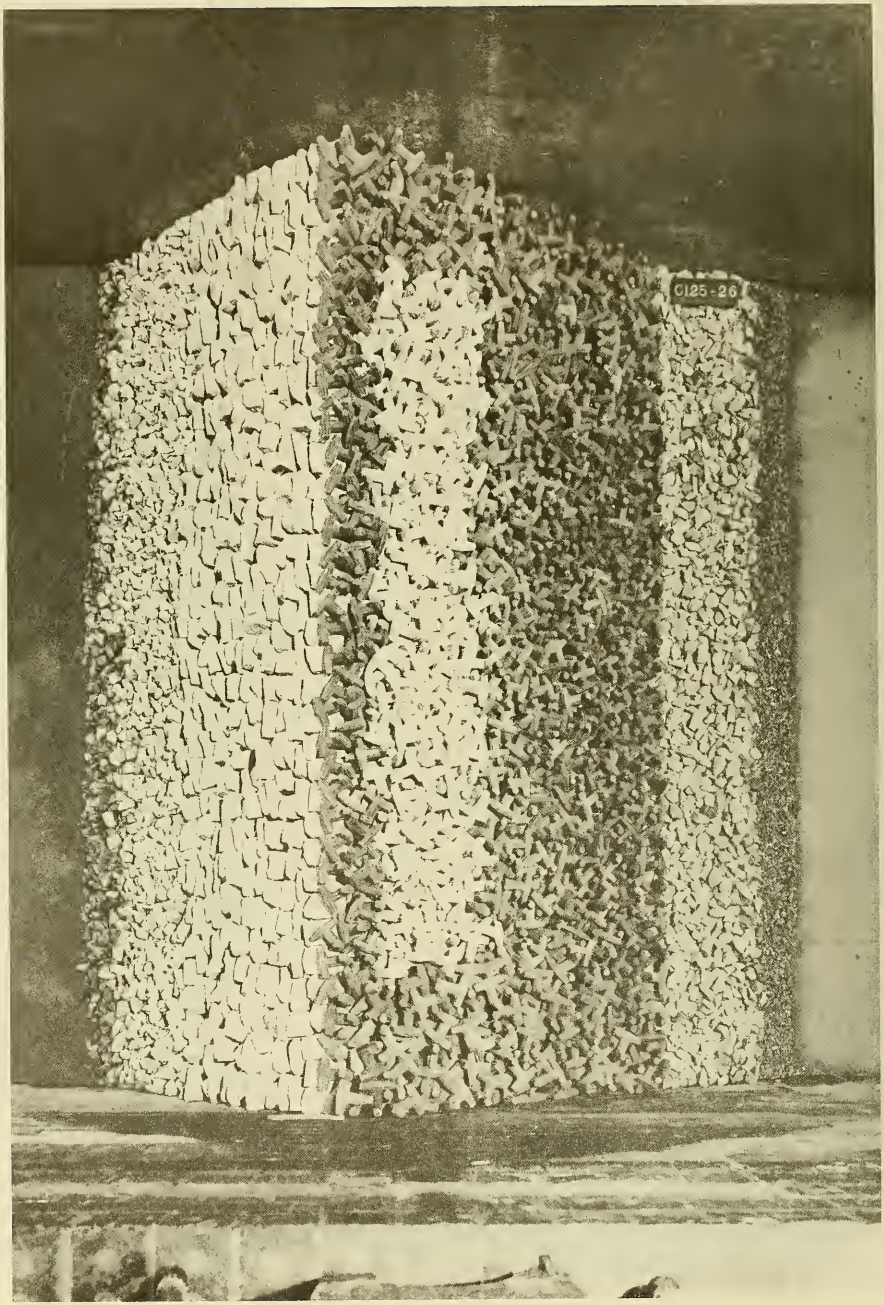


Photo 24. Overhead view of Plan 2 before testing, first test section

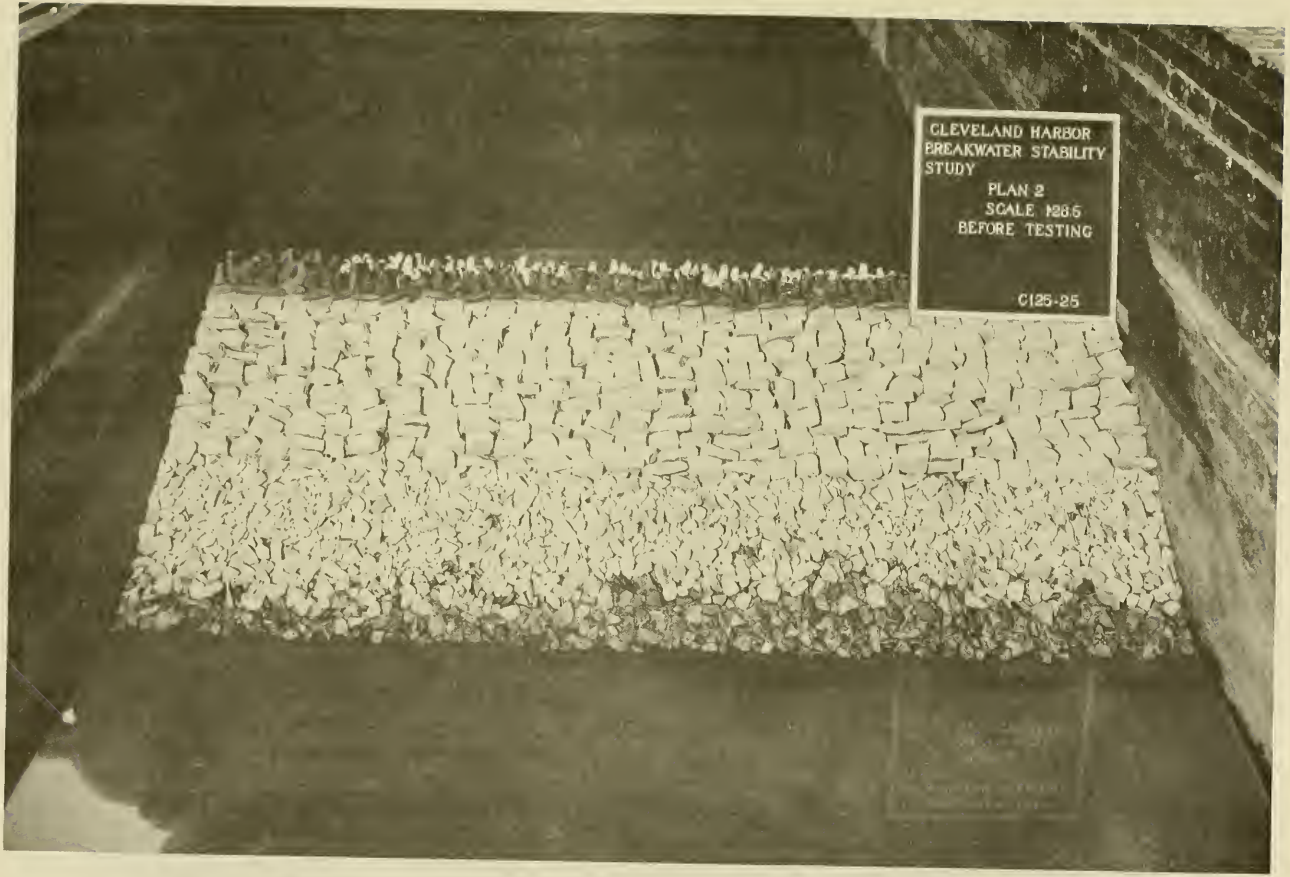


Photo 25. Harbor-side view of Plan 2 before testing, first test section

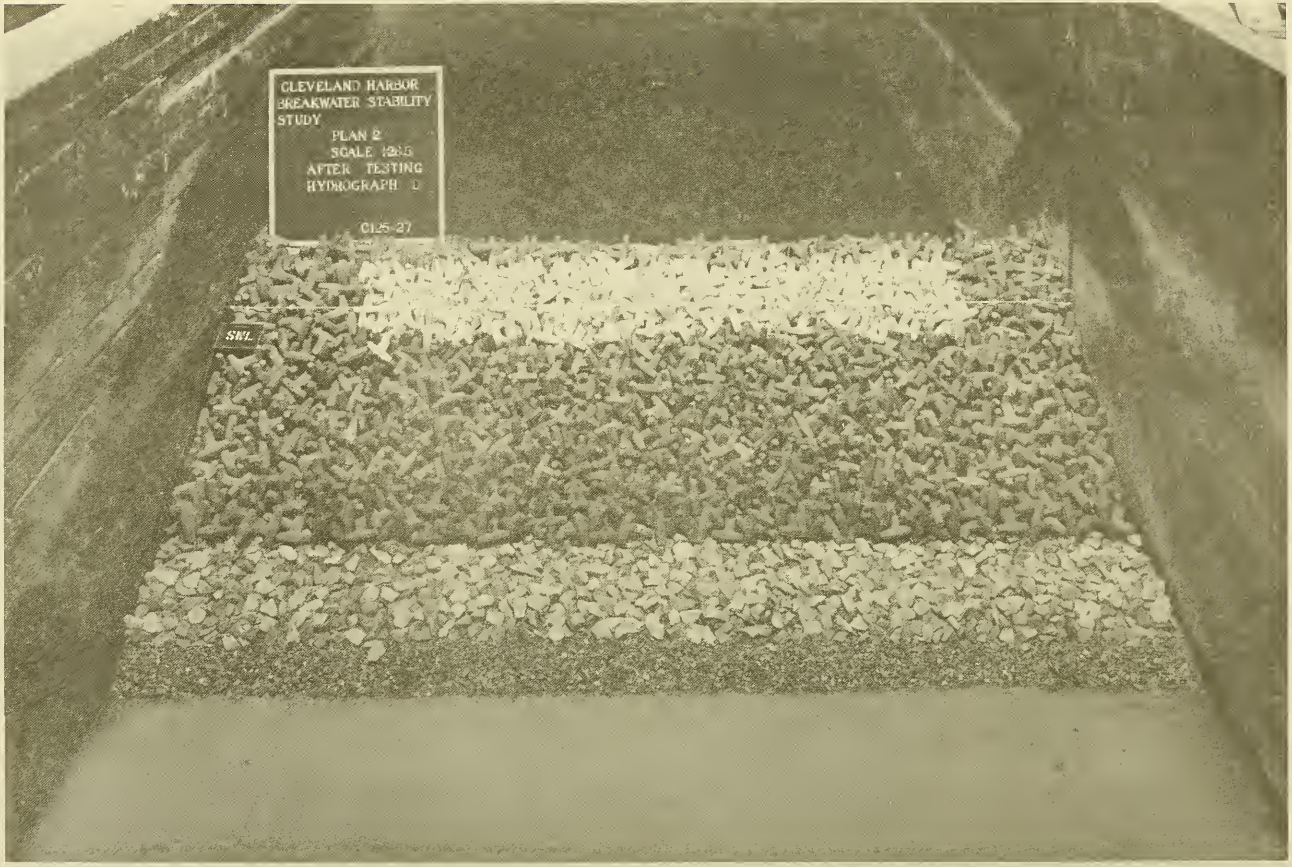


Photo 26. Lakeside view of Plan 2 after testing Hydrograph D, first test section

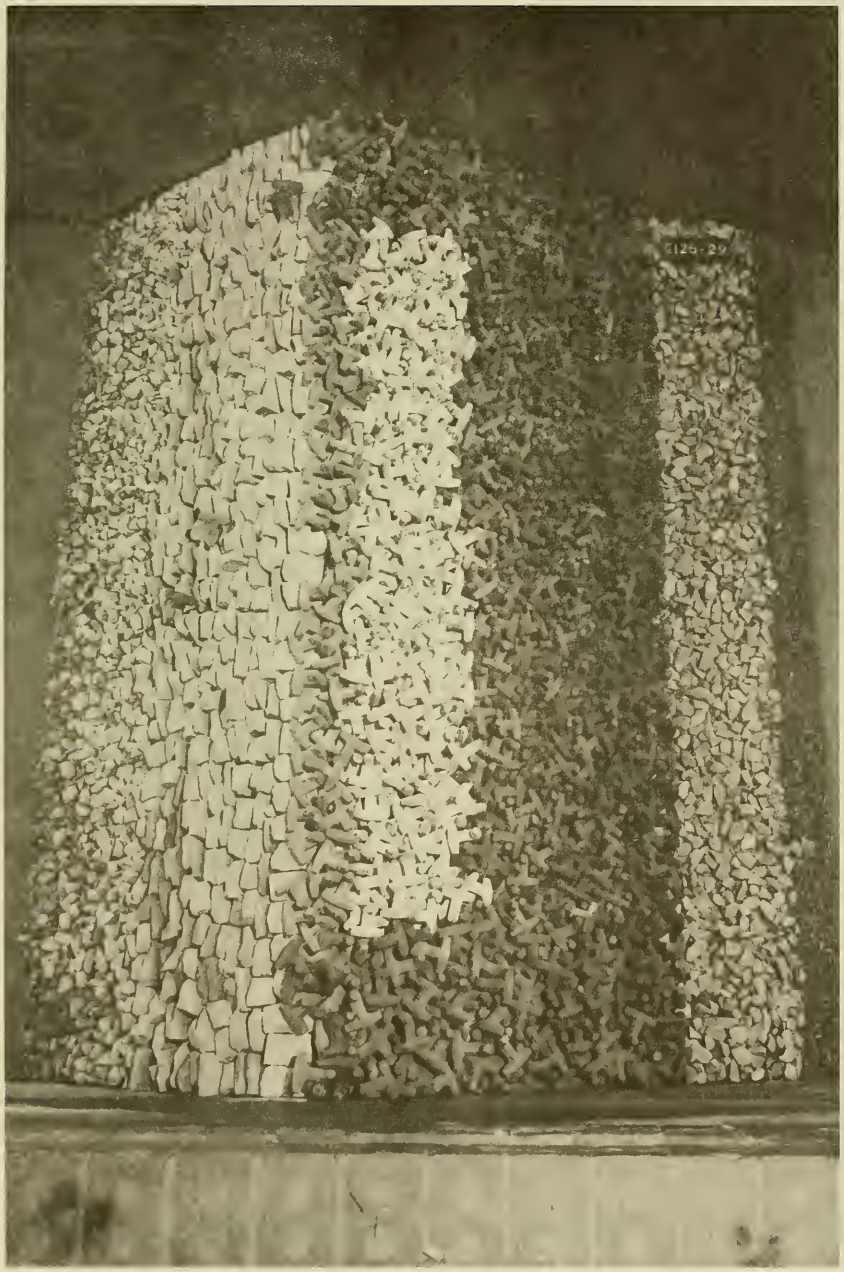


Photo 27. Overhead view of Plan 2 after testing Hydrograph D, first test section

Photo 28. Harbor-side view of Plan 2 after testing Hydrograph D, first test section

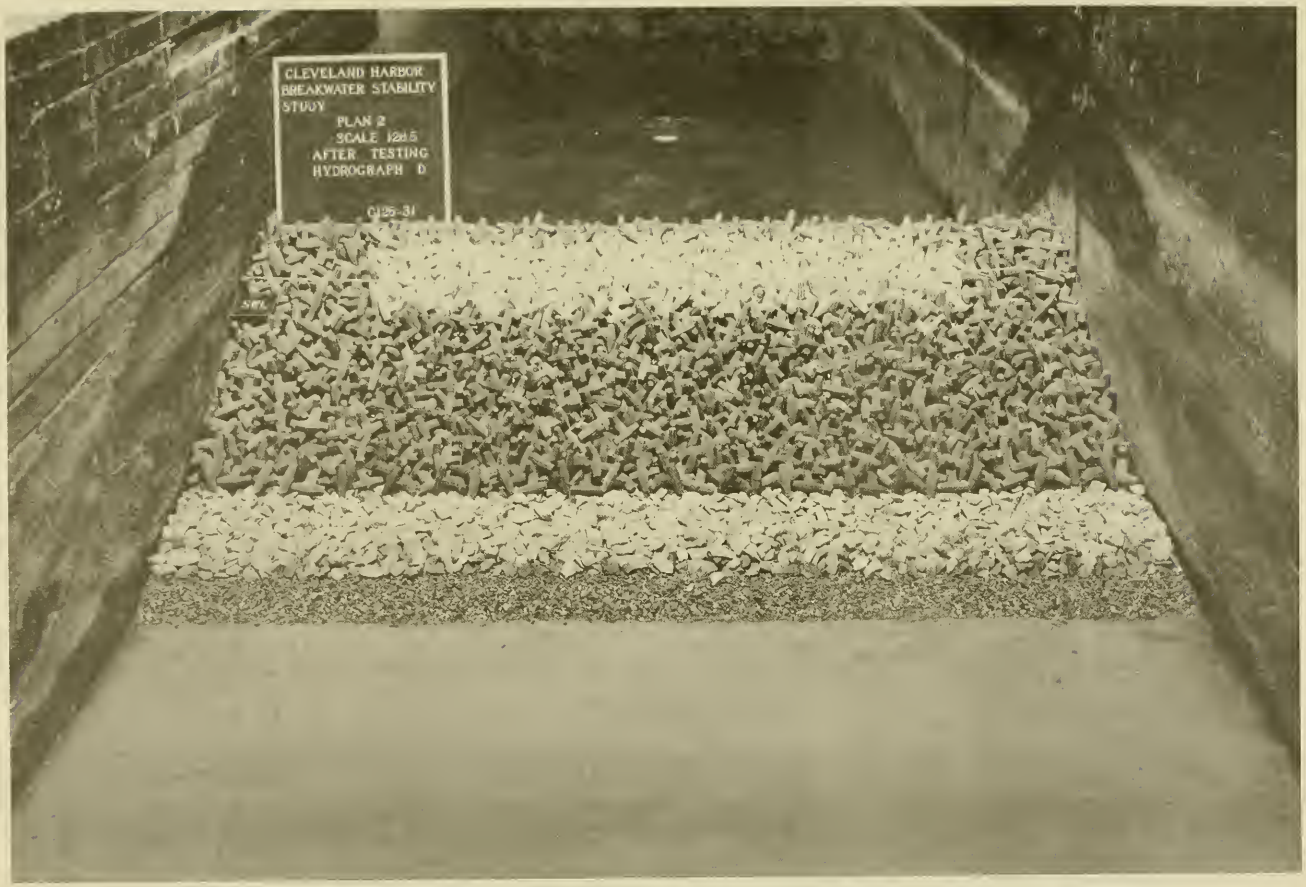


Photo 29. Lakeside view of Plan 2 after testing Hydrograph D, second test section

Photo 30. Overhead view of Plan 2 after testing Hydrograph D, second test section

Photo 31. Harbor-side view of Plan 2 after testing Hydrograph D, second test section

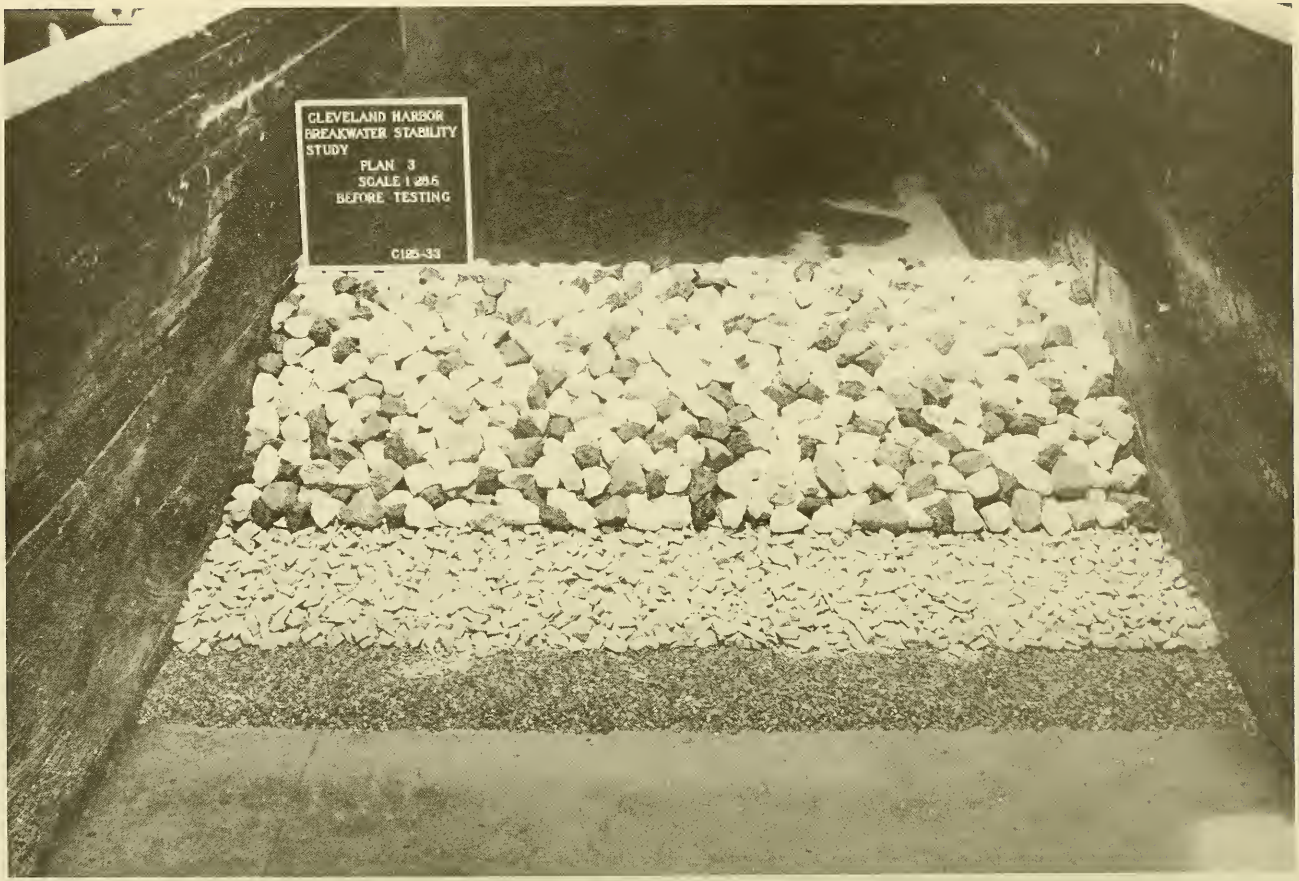


Photo 32. Lakeside view of Plan 3 before testing, first test section

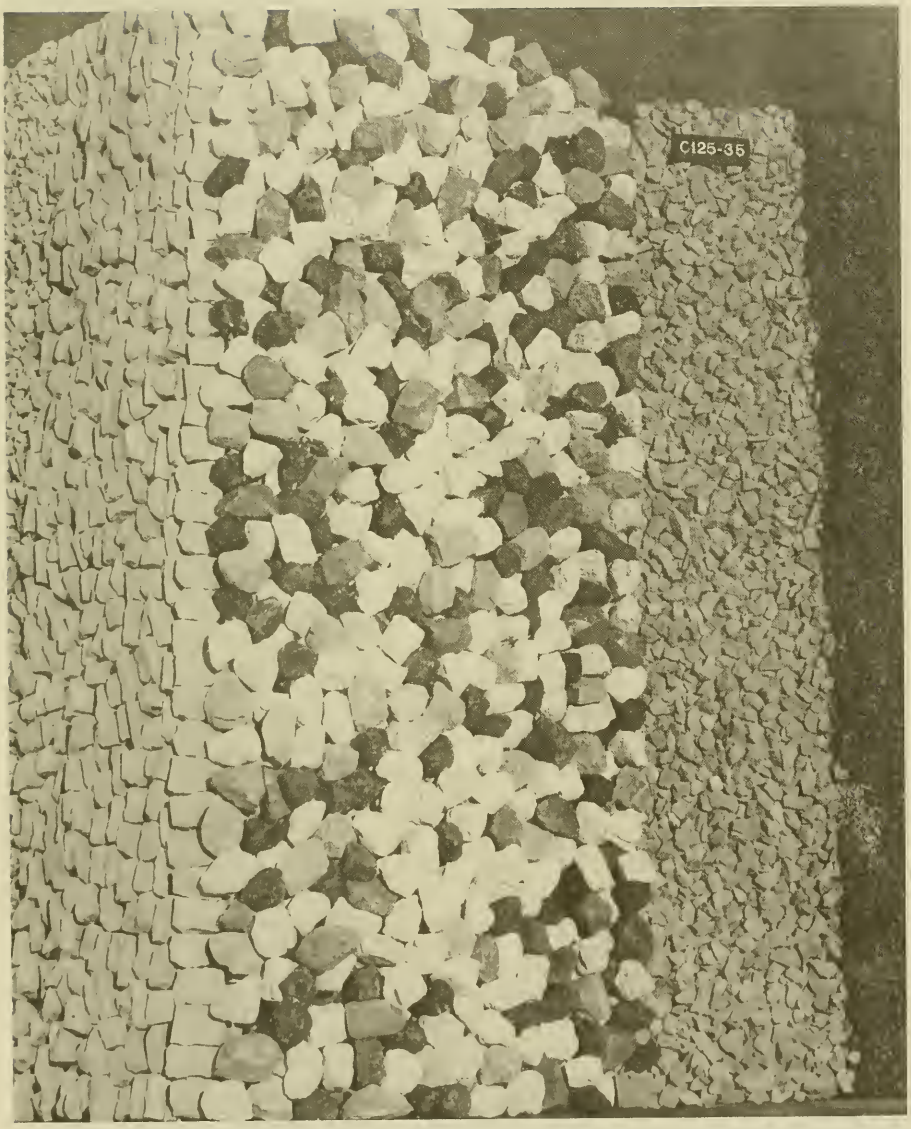


Photo 33. Overhead view of Plan 3 before testing, first test section

Photo 34. Harbor-side view of Plan 3 before testing, first test section

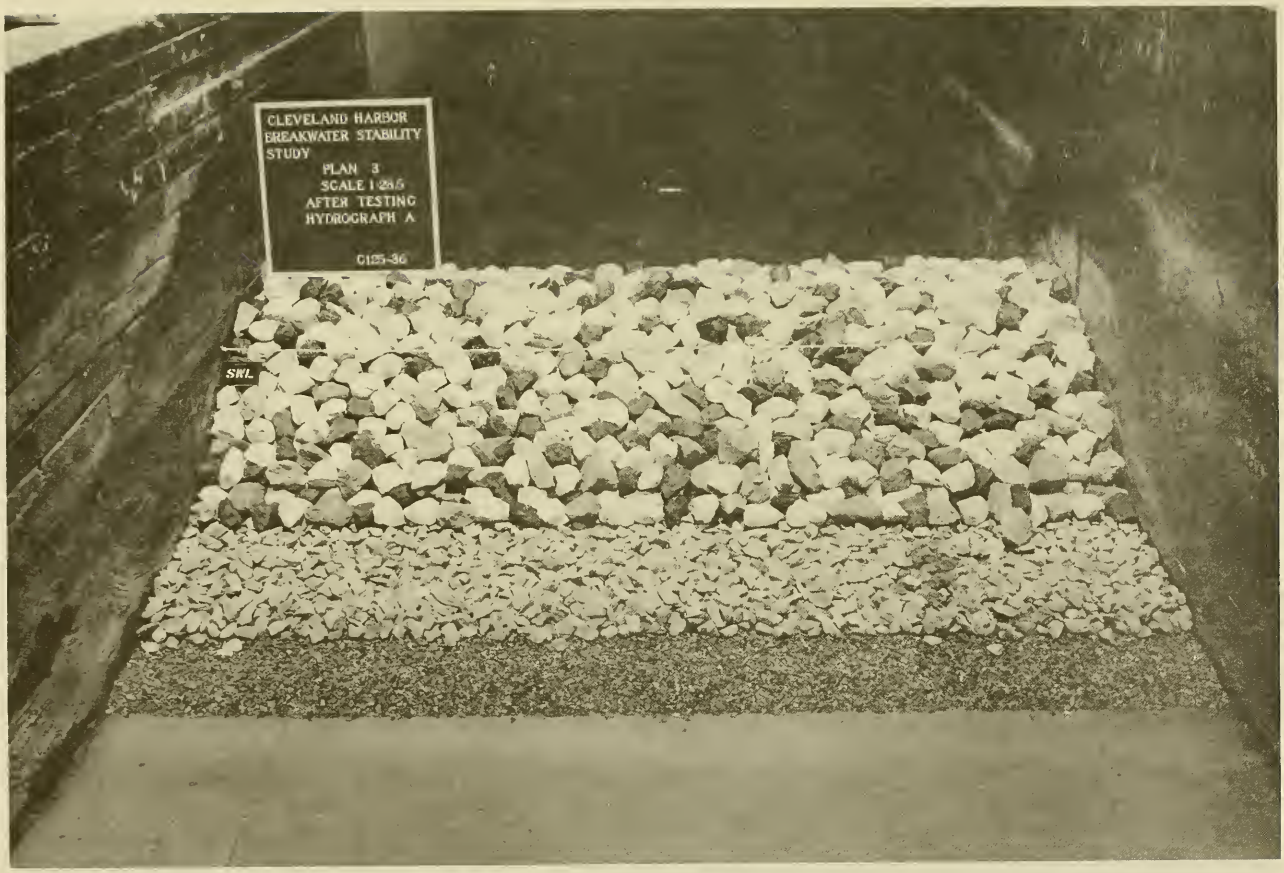


Photo 35. Lakeside view of Plan 3 after testing Hydrograph A, first test section

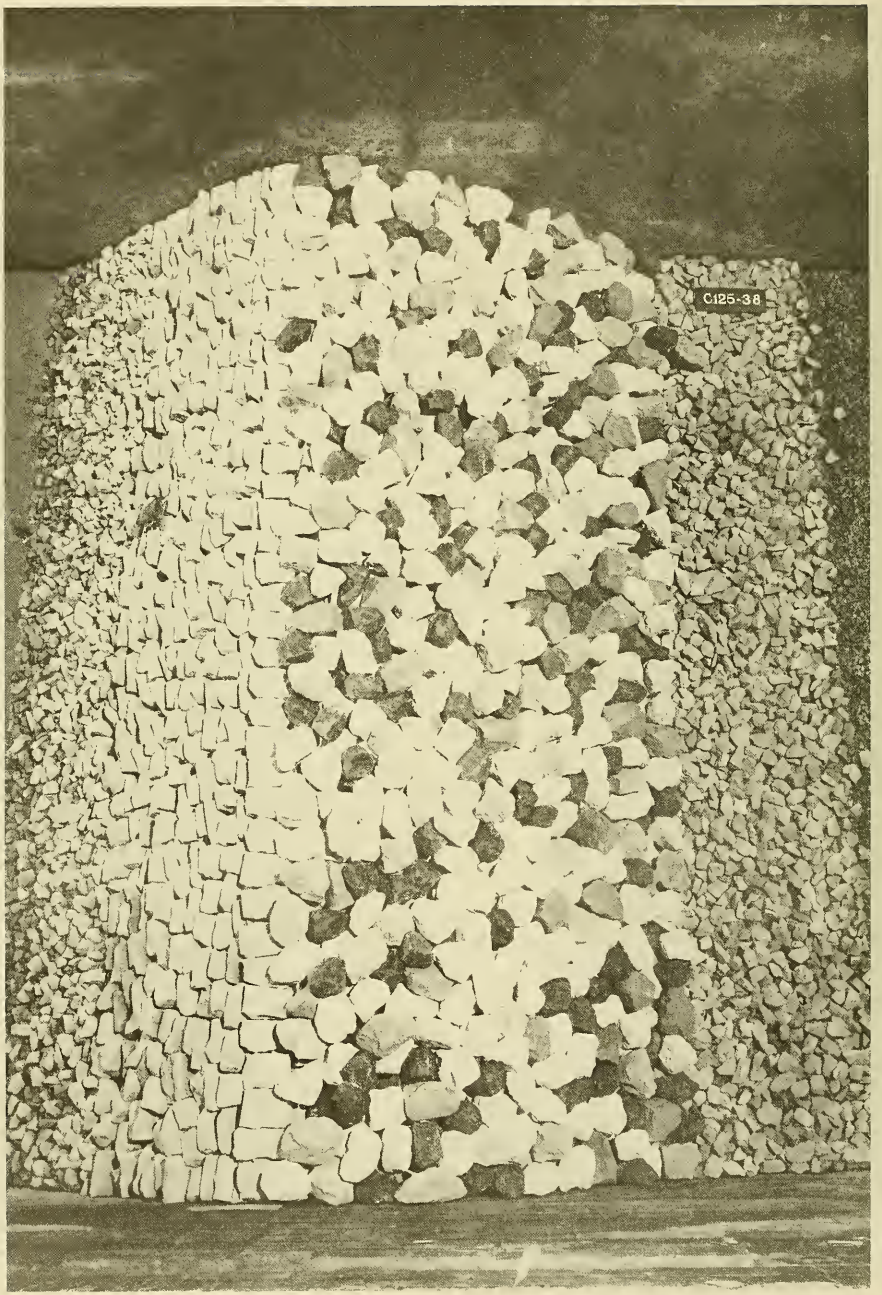


Photo 36. Overhead view of Plan 3 after testing Hydrograph A, first test section

Photo 37. Harbor-side view of Plan 3 after testing Hydrograph A, first test section

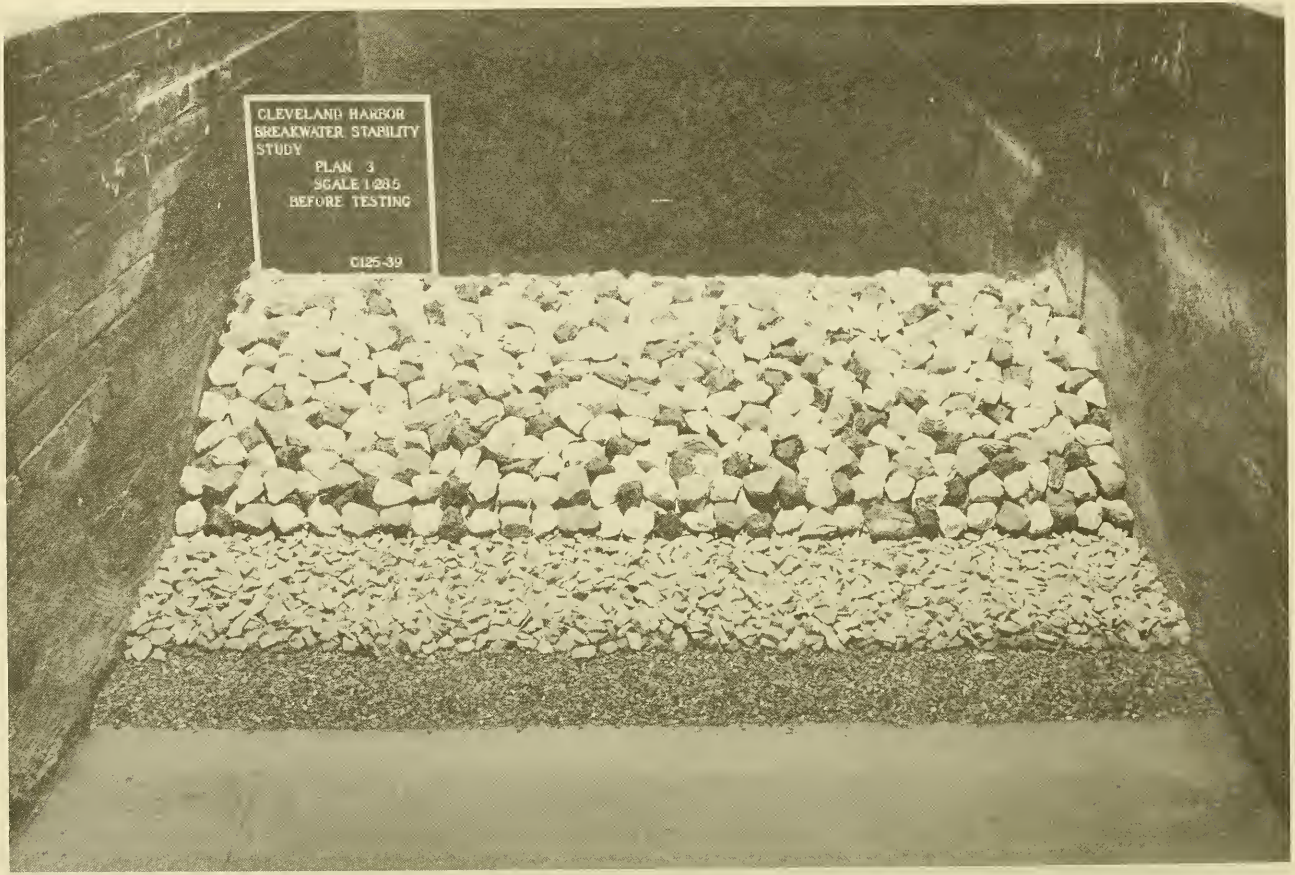


Photo 38. Lakeside view of Plan 3 before testing, second test section

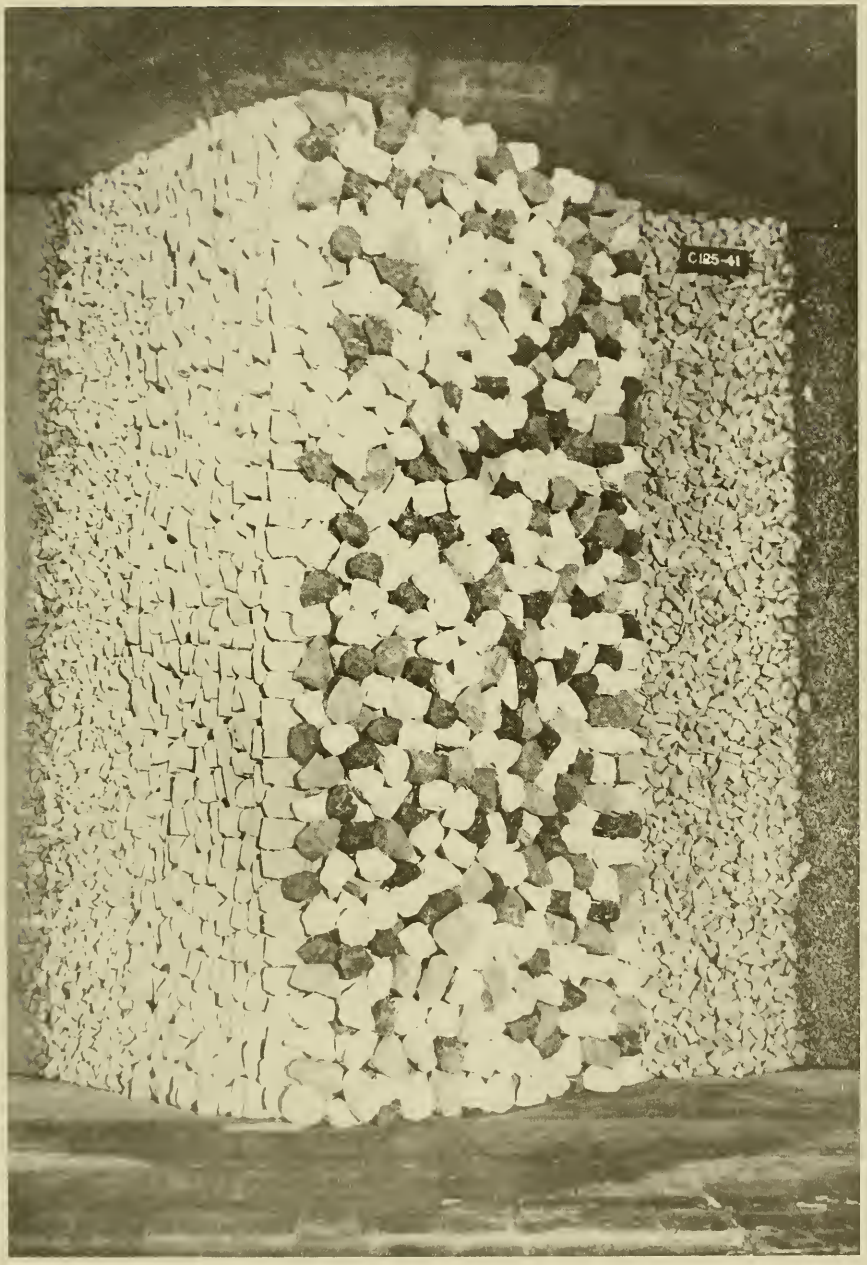


Photo 39. Overhead view of Plan 3 before testing, second test section

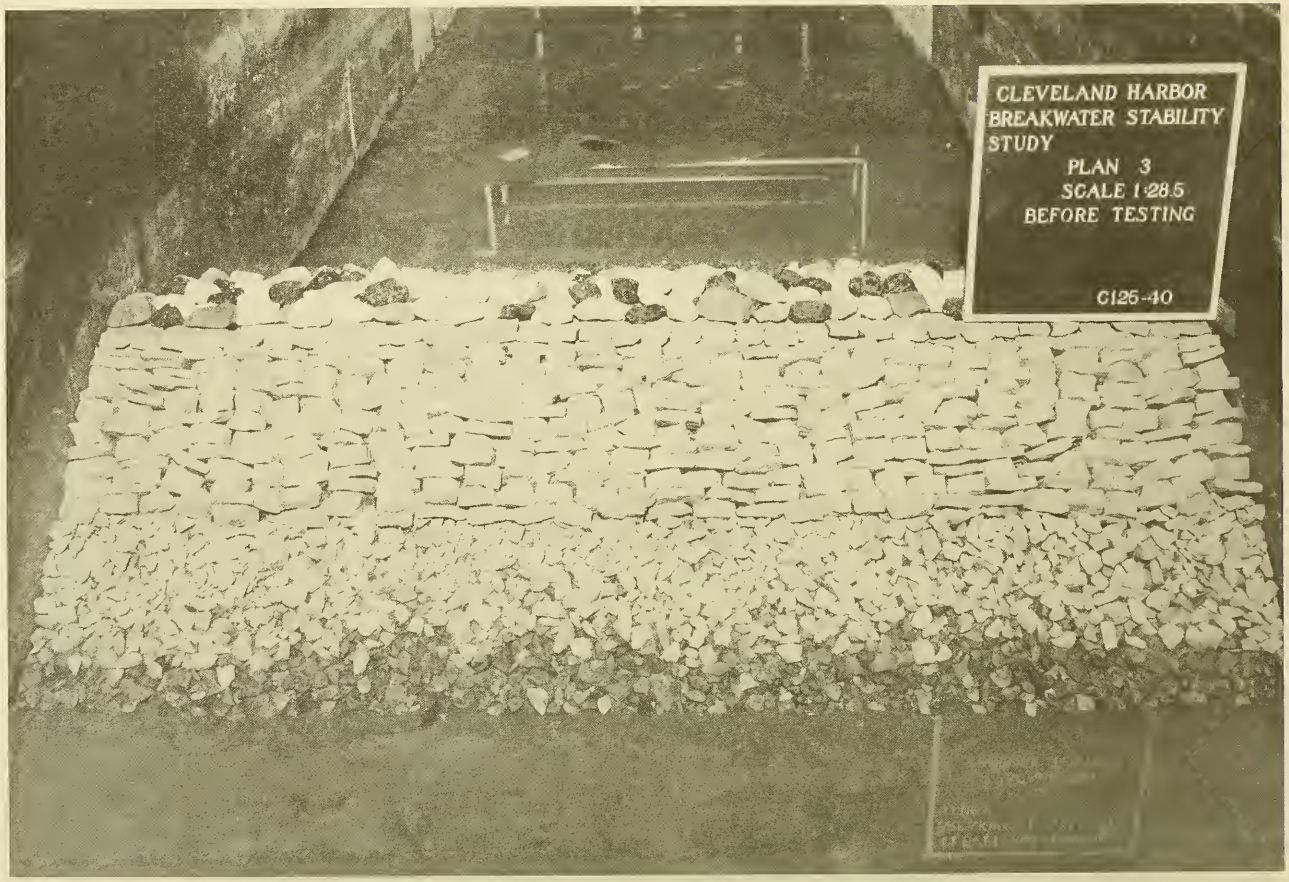


Photo 40. Harbor-side view of Plan 3 before testing, second test section

Photo 41. Lakeside view of Plan 3 after testing Hydrograph A, second test section

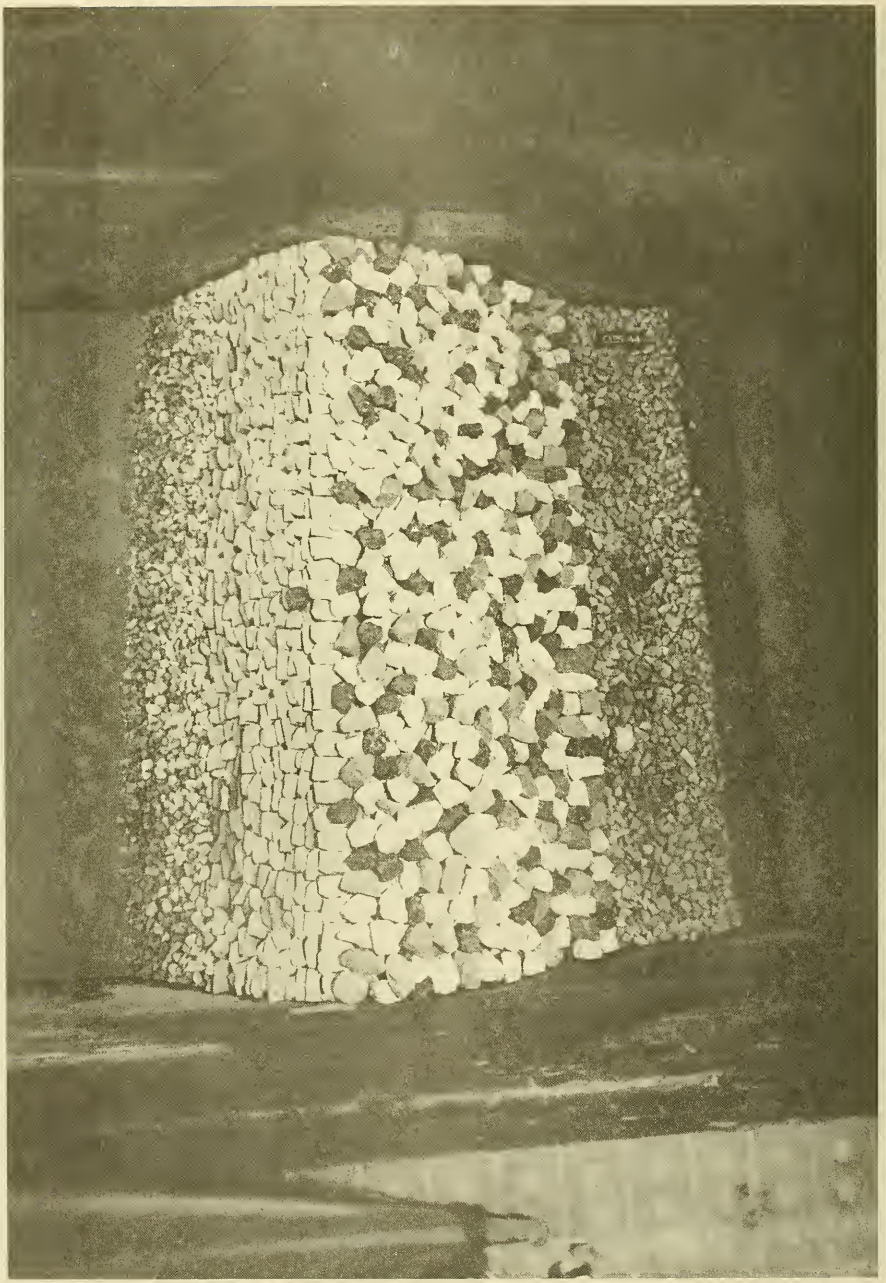
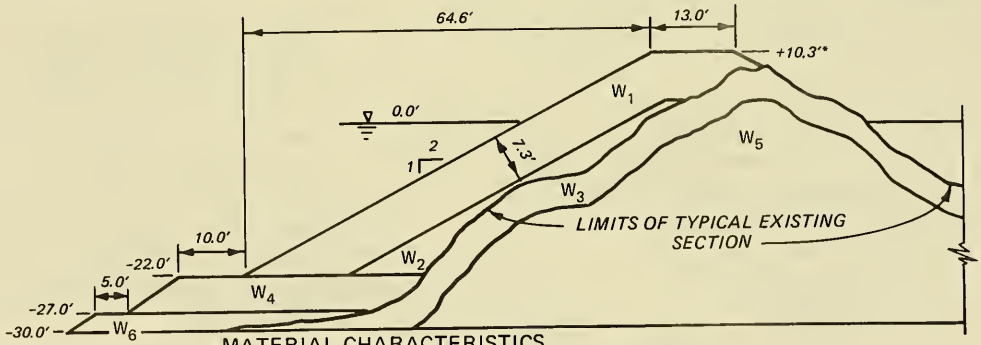


Photo 42. Overhead view of Plan 3 after testing Hydrograph A, second test section

Photo 43. Harbor-side view of Plan 3 after testing Hydrograph A, second test section

LAKESIDE

HARBOR SIDE



MATERIAL CHARACTERISTICS

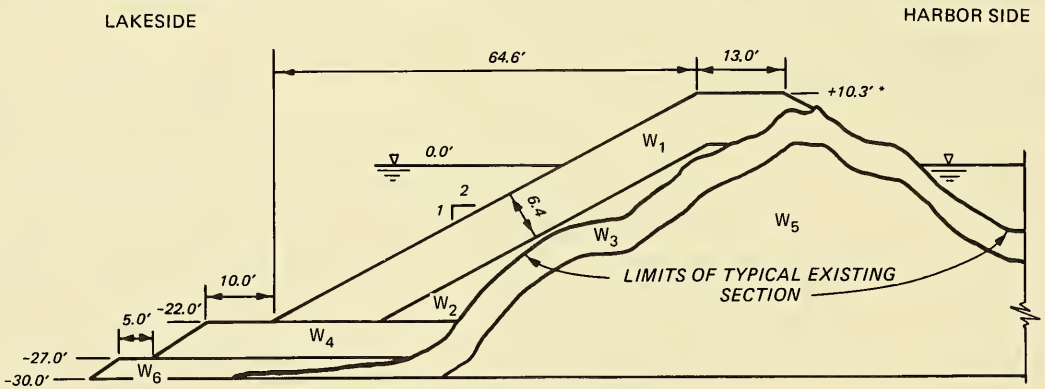
	MODEL	PROTOTYPE
**,†	W_1 = 0.290-LB DOLOSSE @ 144.4 PCF	W_1 = 8,000-LB DOLOSSE @ 140 PCF
**	W_2 = 0.037- TO 0.071-LB STONE @ 165 PCF	W_2 = 1,100- TO 2,100-LB STONE @ 155 PCF
††	W_3 = 0.202- TO 0.541-LB STONE @ 165 PCF	W_3 = 6,000- TO 16,000-LB STONE @ 155 PCF
**	W_4 = 0.044- TO 0.135-LB STONE @ 165 PCF	W_4 = 1,300- TO 4,000-LB STONE @ 155 PCF
	$W_5 \leq$ 0.135-LB STONE @ 165 PCF	$W_5 \leq$ 4,000-LB STONE @ 155 PCF
**	$W_6 \leq$ 0.001-LB STONE @ 165 PCF	$W_6 \leq$ 60-LB STONE @ 155 PCF

* ELEVATIONS IN FEET REFERRED TO LWD
** REHABILITATION MATERIALS
† TWO LAYERS; RANDOM PLACEMENT
†† ONE LAYER; LAID UP PLACEMENT

REHABILITATION SECTION
PLAN 1
4-TON DOLOSSE

PLATE 1

PLATE 2



MATERIAL CHARACTERISTICS

MODEL

**,† W_1 = 0.234-LB DOLOSSE @ 137 PCF

** W_2 = 0.051 - TO 0.028-LB STONE @ 165 PCF

†† W_3 = 0.202 - TO 0.541-LB STONE @ 165 PCF

** W_4 = 0.044 - TO 0.135-LB STONE @ 165 PCF

$W_5 \leq$ 0.135-LB STONE @ 165 PCF

** $W_6 \leq$ 0.001-LB STONE @ 165 PCF

PROTOTYPE

W_1 = 4,600-LB DOLOSSE @ 140 PCF

W_2 = 1,192 - TO 641-LB STONE @ 155 PCF

W_3 = 6,000 - TO 16,000-LB STONE @ 155 PCF

W_4 = 1,300 - TO 4,000-LB STONE @ 155 PCF

$W_5 \leq$ 4,000-LB STONE @ 155 PCF

$W_6 \leq$ 60-LB STONE @ 155 PCF

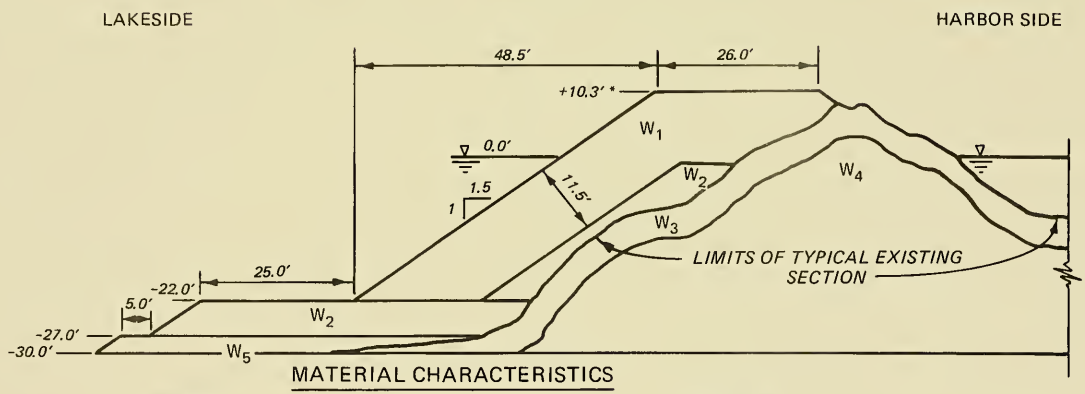
* ELEVATIONS IN FEET REFERRED TO LWD

** REHABILITATION MATERIALS

† TWO LAYERS; RANDOM PLACEMENT

†† ONE LAYER; LAID UP PLACEMENT

REHABILITATION SECTION
PLAN 2
2.3-TON DOLOSSE



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
**,† W_1 = 0.61- TO 1.35-LB STONE @ 165 PCF	W_1 = 18,000- TO 40,000-LB STONE @ 155 PCF
** W_2 = 0.044- TO 0.135-LB STONE @ 165 PCF	W_2 = 1,300- TO 4,000-LB STONE @ 155 PCF
†† W_3 = 0.202- TO 0.541-LB STONE @ 165 PCF	W_3 = 6,000- TO 16,000-LB STONE @ 155 PCF
$W_4 \leq$ 0.135-LB STONE @ 165 PCF	$W_4 \leq$ 4,000-LB STONE @ 155 PCF
** $W_5 \leq$ 0.001-LB STONE @ 165 PCF	$W_5 \leq$ 60-LB STONE @ 155 PCF

* ELEVATION IN FEET REFERRED TO LWD
** REHABILITATION MATERIALS
† TWO LAYERS; RANDOM PLACEMENT
†† ONE LAYER; LAID UP PLACEMENT

REHABILITATION SECTION
PLAN 3
9-TO 20-TON ARMOR STONE

PLATE 3